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Engineering Summary Appendix

Shasta Lake Water Resources Investigation

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Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
AB	aggregate base
AC	asphaltic concrete
APS	allowance for procurement strategies
AREMA	American Railway Engineering and Maintenance-of–Way Association
ASTM	American Society for Testing and Materials
AT&T	American Telephone and Telegraph
AWWA	American Water Works Association
BIL	basic impulse level
BLM	U.S. Department of the Interior, Bureau of Land Management
Caltrans	California Department of Transportation
cfs	cubic foot per second
CIP	cast in place
CMP	corrugated metal pipe
CP	Comprehensive Plan
CPUC	California Public Utilities Commission
CPUC-GO 95	California Public Utilities Commission General Order 95
CSA	County Service Area
CVP	Central Valley Project
CVPIA	Central Valley Project Improvement Act
DBH	diameter at breast height
Delta	Sacramento-San Joaquin Delta
DFG	California Department of Fish and Game
EHD	Environmental Health Division
elevation xxx	elevation in feet above mean sea level
EPA	Environmental Protection Agency
ESRD	Emergency Spillway Release Diagram
gpd	gallon per day
gpm	gallon per minute
HHA	hydrologic hazard analysis
HMR 59	Hydrometeorological Report No. 59
km	kilometer

kV	kilovolt
L-G voltage	voltage between phase conductor and ground
M&I	municipal and industrial
MAF	million acre-feet
mm	millimeter
MSE	mechanically stabilized earth
msl	mean sea level
MW	megawatt
NAVD88	North American Vertical Datum of 1988
NCAO	Northern California Area Office
NESC	National Electrical Safety Code
NESHAP	National Emission Standards of Hazardous Air Pollutants
NFPA	National Fire Protection Association
NGS	National Geodetic Survey
NGVD29	National Geodetic Vertical Datum of 1929
NRA	National Recreation Area
PFR	Plan Formulation Report
PG&E	Pacific Gas and Electric
PMF	probable maximum flood
psi	pound per square inch
PVC	polyvinyl chloride
RBDD	Red Bluff Diversion Dam
Reclamation	U.S. Department of the Interior, Bureau of Reclamation
ROW	right-of-way
RWS	reservoir water surface
SDR	standard dimension ratio
SLWRI	Shasta Lake Water Resources Investigation
SWP	State Water Project
TCD	temperature control device
UPRR	Union Pacific Railroad
USFS	U.S. Forest Service
USGS	U.S. Geological Survey
UV	ultraviolet
V	volt

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Chapter 1
Introduction

Purpose and Need for Project

The primary purpose of the Shasta Lake Water Resources Investigation (SLWRI) is to develop an implementable plan that primarily involves modifying Shasta Dam and Reservoir to promote increased survival of anadromous fish populations in the upper Sacramento River; promote increased water supplies and water supply reliability; and to the greatest extent possible through meeting these primary planning objectives, include features to benefit other identified ecosystem, flood control, and related water resources needs.

Background

The following sections describe the study area location, study authorization, and the scope, purpose, and organization of this Engineering Summary Appendix.

Study Area Location and Description

The primary study area for the SLWRI includes Shasta Dam and Reservoir, lower reaches of three primary tributaries flowing into Shasta Lake (Sacramento, McCloud, and Pit rivers) and all smaller tributaries flowing into the lake, Trinity Lake and Lewiston Reservoir, and the Sacramento River downstream to about the Red Bluff Diversion Dam (RBDD). Plate 1 is a vicinity map showing the primary study area within the Sacramento River basin. The RBDD is the point at which releases from Shasta Dam begin to have a negligible effect on Sacramento River water temperatures, and the river landscape changes to a broader, alluvial stream system.

Because of the potential influence of a modified Shasta Dam on other programs and projects, primarily in the Central Valley, an extended study area also encompasses the Sacramento River downstream from the RBDD, the Sacramento-San Joaquin River Delta (Delta), parts of the lower American and Feather rivers, parts of the lower San Joaquin River; and facilities and water service areas of the Central Valley Project (CVP) and State Water Project (SWP).

Shasta Dam and Reservoir are located on the upper Sacramento River in Northern California about 9 miles northwest of the City of Redding (see Plate 1); the entire reservoir is within Shasta County. Shasta Lake has 370 miles of shoreline. The reservoir controls runoff from about 6,420 square miles. The four major tributaries to Shasta Lake are the Sacramento River, McCloud River, Pit River, and Squaw Creek, in addition to numerous minor tributary creeks and streams.

Study Authorization

On August 30, 1935, in the Rivers and Harbors Bill, an initial amount of Federal funding was authorized for constructing Kennett (now Shasta) Dam. Fundamental authorization for the SLWRI derives from the 1980 Public Law 96-375 and 2004 Public Law 108-361. Public Law 96-375 authorized the Secretary of the Interior to engage in feasibility studies relating to (1) enlarging Shasta Dam and Reservoir, or constructing a replacement dam on the Sacramento River, and (2) using the Sacramento River to convey water from an enlarged dam. Public Law 108-361 again directed the Secretary of the Interior to conduct "...planning and feasibility studies for projects to be pursued with project-specific study for enlargement of…Shasta Dam in Shasta County…"

Scope and Purpose of Engineering Summary Appendix

The primary purpose of this Engineering Summary Appendix is to present information related to feasibility-level cost estimates and designs for the measures included in the comprehensive plans described in the SLWRI Draft Feasibility Report (Reclamation 2011c). The measures included in each of the comprehensive plans can be put into three categories: dam raises, reservoir area infrastructure, and ecosystem restoration. Information associated with these measures will be used to compare the comprehensive plans. Previous SLWRI milestone documents were used as an initial basis for development of feasibility-level designs and cost estimates for this Engineering Summary Appendix.

Appendix Organization

This Engineering Summary Appendix is organized as follows:

Chapter 1 introduces the SLWRI, provides background on the study, and describes the scope, purpose, and organization of this appendix.

Chapter 2 provides background information on Shasta Dam and Reservoir and describes the three dam raise and reservoir enlargement options included in the comprehensive alternatives presented in the SLWRI Draft Feasibility Report (Reclamation 2011c). These alternatives include the 6.5-foot raise, 12.5-foot raise, and 18.5-foot raise.

Chapter 3 describes design considerations for the dam and appurtenances raise options.

Chapter 4 describes design considerations for reservoir area infrastructure modifications and/or relocations for the raise options.

Chapter 5 presents cost estimates developed for each of the comprehensive plans, and information and methodology used to develop the estimates.

Chapter 6 contains sources used to prepare this Engineering Summary Appendix.

Summary and detailed cost estimate worksheets are included in the following attachments to this Engineering Summary Appendix:

Attachment 1 – Cost Estimates for Comprehensive Plans

Attachment 2 – 6.5-Foot Raise and Reservoir Area Infrastructure Cost Estimates

Attachment 3 – 12.5-Foot Raise and Reservoir Area Infrastructure Cost Estimates

Attachment 4 – 18.5-Foot Raise and Reservoir Area Infrastructure Cost Estimates

Attachment 5 – Preliminary Construction Schedule and Work Packages

Attachment 6 – CP4 Crystal Ball Estimate Summary

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Chapter 2
Dam and Reservoir Raise Options

Shasta Dam and Reservoir Background

Shasta Reservoir is California's largest man-made lake, with a full pool storage capacity and surface area at the top of joint-use of 4,552,000 acre-feet and 29,600 acres, respectively. (Top of joint-use is measured at elevation 1,067 feet above mean sea level (msl) (elevation 1,067) according to the National Geodetic Vertical Datum of 1929 (NGVD29).) As mentioned, Shasta Reservoir has approximately 370 miles of shoreline when full, and a maximum depth of 517 feet. The Shasta Dam and Reservoir project was constructed by the U.S. Department of the Interior, Bureau of Reclamation, Region 2 (Reclamation), as an integral element of the CVP from 1938 to 1945, for six purposes. These purposes include irrigation water supply, municipal and industrial (M&I) water supply, flood control, hydropower generation, fish and wildlife conservation, and navigation. The project also supports vigorous water-oriented recreation at the reservoir, which is located within the Shasta Unit of the Whiskeytown-Shasta-Trinity National Recreation Area (NRA). Table 2-1 presents pertinent Shasta Dam and Reservoir data Plate 2 shows the reservoir and numerous surrounding facilities.

Shasta Dam and Reservoir are located on the upper Sacramento River in Northern California about 9 miles northwest of the City of Redding. The entire reservoir is within Shasta County. The reservoir controls runoff from about 6,420 square miles from four major tributaries, including the Sacramento, McCloud, and Pit rivers, Squaw Creek, and numerous minor creeks and streams. Historically, essentially all outflow from Shasta Dam has traveled through Northern California to the Delta southwest of Sacramento. Total drainage area of the Sacramento River at the Delta is about 26,300 square miles. Average annual runoff to the Delta from the Sacramento River watershed is about 17.2 million acre-feet (MAF). This represents about 62 percent of total inflows to the Delta.

Shasta Dam is a curved, gravity-type, concrete structure 487 feet high above the streambed, with a total height above the foundation of 602 feet. Its crest is at elevation 1,077.5 (NGVD29). Maximum seasonal flood control storage space in Shasta Reservoir is 1.3 MAF. Shasta Dam has a crest width of 30 feet and length of about 3,500 feet. The Shasta Powerplant consists of five main generating units with a current capacity of 710 megawatts (MW), and two station service units with a current capacity of 5 MW. Plan views of Shasta Dam and Powerplant are shown in Plate 3. Figure 2-1 shows the area-capacity curve for Shasta Reservoir.

Table 2-1. Pertinent Data – Shasta Dam and Reservoir

General			
Drainage Areas (excluding Goose Lake Basin)		**Mean Annual Runoff (1908 – 2006)**	
Sacramento R. at Shasta Dam	6,421 square miles	Sacramento R. at Shasta Dam	5,737,000 acre-feet
Sacramento R. at Keswick	6,468 square miles	Sacramento R. near Red Bluff	8,421,000 acre-feet
Bridge near Red Bluff	8,900 square miles	**Sacramento River Maximum Flows**	
Sacramento R. near Ord Ferry	12,250 square miles	At Shasta Lake 16 Jan 1974	216,000 cfs
Pit R. at Big Bend	4,710 square miles	Near Red Bluff 28 Feb 1940	291,000 cfs
McCloud R. above Shasta Lake	604 square miles	At Ord Ferry 28 Feb 1940	370,000 cfs
Sacramento R. at delta above Shasta Lake	425 square miles		
Shasta Dam and Reservoir			
Shasta Dam (concrete gravity)		**Shasta Reservoir**	
Crest elevation	1,077.5 feet	Full pool elevation (msl)	1,067.0 feet
Freeboard above full pool	9.5 feet	Minimum operating level	840.0 feet
Height above foundations	602 feet	Taking line	Irregular
Height above streambed	487 feet	Surface Area	
Length of crest	3,500 feet	Minimum operating level	6,700 acres
Width of crest	30 feet	Full pool	29,500 acres
Slope, upstream	Vertical	Taking line	90,000 acres
Slope, downstream	1 on 0.8	Storage capacity	
Volume	8,430,000 cubic yards	Minimum operating level	587,000 acre-feet
Normal tailwater elevation	585 feet	Full pool	4,552,000 acre-feet
Spillway (gated ogee)		**Shasta Powerplant**	
Crest length		Main units	
Gross	360 feet	5 turbines, Francis type	515,000 hp (total)
Net	330 feet	5 units @ 142 MW	710 MW (total)
Crest gates (drum type)		Station units	
Number and size	3 @ 110 feet x 28 feet	2 generators, 2,500 kW each	5,000 kW (total)
Top elevation when lowered	1037.0 feet	Elevation centerline turbines	586 feet
Top elevation when raised	1065.0 feet	Maximum tailwater elevation	632.5 feet
Discharge capacity at pool (1,065 feet)	186,000 cfs	Total discharge at pool (1,065 feet)	14,500 cfs

Table 2-1. Pertinent Data – Shasta Dam and Reservoir (contd.)

Shasta Dam and Reservoir (contd.)			
Drainage Areas (excluding Goose Lake Basin)		**Mean Annual Runoff (1908 – 2006)**	
Flashboard gates	3 @ 110 feet x 2 feet	Total discharge at pool (827.7 feet)	16,000 cfs
Top elevation when lowered	1,067.0 feet	Power outlets (15-foot steel penstocks)	
Bottom elevation when raised	1,069.5 feet	5 with invert elev. of intake	807.5 feet
Outlets 102-inch-diameter conduit with 96-inch-diameter wheel-type gate			
4 with invert elevation	737.75 feet	Capacity at elevation 1,065 feet	81,800 cfs
8 with invert elevation	837.75 feet	Capacity at elevation 827.7 feet	12,200 cfs
6 with invert elevation	937.75 feet		

Note:
Elevations given are in vertical datum NGVD 1929.
Key:
cfs = cubic feet per second
elevation = elevation in feet above mean sea level
hp = horsepower
kW = kilowatt
msl = mean sea level
MW = megawatt
R = river

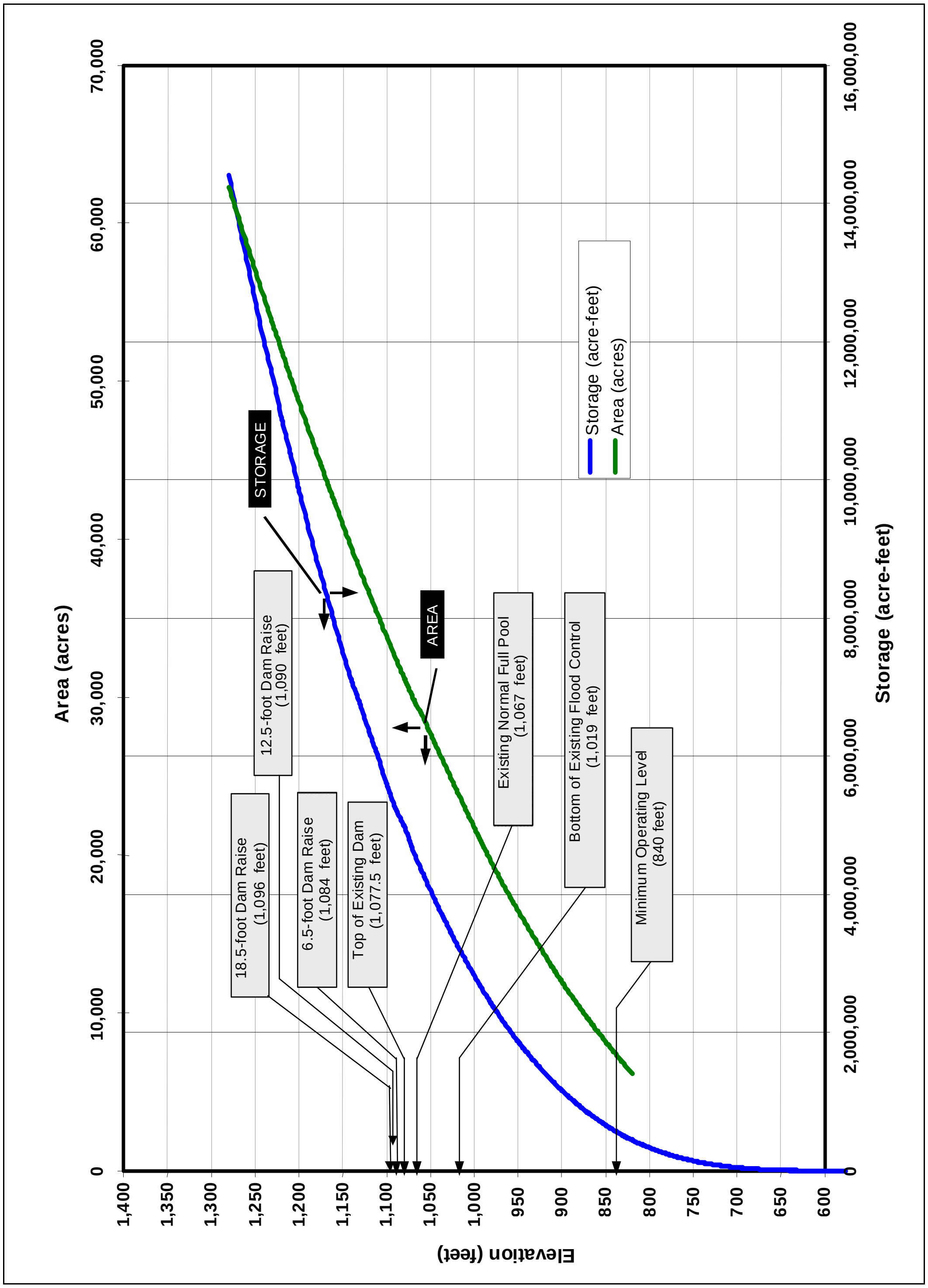

Figure 2-1. Area-Capacity Curve for Shasta Reservoir

Vertical Datum Differences

Currently, elevations listed in most reference materials related to Shasta Dam and Reservoir are in NGVD29. However, a 2001 aerial survey of the reservoir area was done using the North American Vertical Datum of 1988 (NAVD88). All current feasibility designs and plates for the dam and appurtenances are based on NGVD29. All current feasibility designs and plates for reservoir area infrastructure modifications and/or relocations are based on the 2001 aerial survey and use NAVD88, unless otherwise noted. These include the designs and plates of the protective dikes in the upstream reservoir.

According to the National Geodetic Survey (NGS) Program *VERTCON*, the difference between the vertical datums at Shasta Dam is 2.664 feet (0.812 meters). Table 2-2 lists key elevations in both vertical datums for comparison and clarification.

Table 2-2. Vertical Datum Comparison

Feature	Existing (feet)	6.5-foot Dam Raise (feet)	12.5-foot Dam Raise (feet)	18.5-foot Dam Raise (feet)
Vertical Datum: NGVD29				
Dam Crest	1,077.5	1,084.0	1,090.0	1,096.0
Full Pool/Top of Joint-Use	1,067.0	1,075.5	1,081.5	1,087.5
Spillway Crest	1,037.0	1,037.5	1,0433.5	1,049.5
Vertical Datum: NAVD88				
Dam Crest	1,080.2	1,086.7	1,092.7	1,098.7
Full Pool/Top of Joint-Use	1,069.7	1,078.2	1,084.2	1,090.2
Spillway Crest	1,039.7	1,040.2	1,046.2	1,052.2

Key:
NAVD88 = North American Vertical Datum of 1988
NGVD29 = National Geodetic Vertical Datum of 1929

Dam Raise and Reservoir Enlargement Options

The proposed comprehensive plans (CP) can be categorized into three dam raise options: 6.5 feet, 12.5 feet, and 18.5 feet. Each comprehensive plan includes, to some degree, the following common measures:

- Enlarge Shasta Lake cold-water pool
- Modify temperature control device (TCD)
- Increase water conservation storage
- Reduce water demand
- Modify flood control operations
- Modify hydropower facilities

- Maintain or increase recreation opportunities
- Maintain or improve water quality

6.5-Foot Raise Option (CP1)

The low-level option evaluated is a 6.5-foot raise of Shasta Dam. This would correspond to a raise in the top of joint-use pool elevation of about 8.5 feet, and increasing water storage capacity by approximately 256,000 acre-feet and reservoir surface area by 1,110 acres. This would increase water supply reliability and improve anadromous fish survival, with some benefit to other resources.

12.5-Foot Raise Option (CP2)

The 12.5-foot raise is considered in this Engineering Summary Appendix as an intermediate-level option between the 6.5-foot raise and the 18.5-foot raise. The 12.5-foot dam raise would correspond to a raise in the top of joint-use pool elevation of about 14.5 feet, increasing the water storage capacity by approximately 443,000 acre-feet and reservoir surface area by 1,750 acres. This would increase water supply reliability and improve anadromous fish survival, with some benefit to other resources.

18.5-Foot Raise Option (CP3, CP4, CP5)

The 18.5-foot raise represents the largest practical dam raise that would not require relocating the Pit River Bridge. This would correspond to a raise in the top of joint-use pool elevation of about 20.5 feet. This option would increase water storage capacity by approximately 634,000 acre-feet and reservoir area by about 2,500 acres. Also, this option would increase water supply reliability and improve anadromous fish survival, with some benefit to other resources. CP3, CP4, and CP5 would all include the 18.5-foot dam raise. However, CP4 would dedicate about 60 percent of the new storage space (378,000 acre-feet) to increasing the cold-water supply for anadromous fish purposes and include features for ecosystem restoration. CP5 would include features for ecosystem restoration and recreation facilities around Shasta Lake, in addition to those included in the other comprehensive plans.

Other Options Considered in Previous Studies

The potential enlargement of Shasta Dam has been studied since the 1970s. In addition to the low-level option (6.5 feet), the 1999 Reclamation Appraisal Assessment also presented descriptions and cost estimates for the intermediate-level option (102.5 feet), and high-level option (202.5 feet) raises of Shasta Dam. Estimated total first costs for the intermediate- and high-level options were $2.9 billion, and $4.4 billion, respectively. The assessment concluded that the costs of the intermediate- and high-level options posed significant challenges in developing required financial packages. Results of the assessment led to the following recommendation: “It is recommended that feasibility studies examining a low-raise option enlargement of Shasta Dam and Reservoir proceed.”

Chapter 3
Design Considerations for Dam and Appurtenances of Dam Enlargements

Dam Crest Structure Removal

Before any enlargement of Shasta Dam, existing structures on the dam crest would need to be removed. These structures include the gantry crane, existing spillway drum gates and frames, spillway bridge, concrete in the spillway crest and abutments, upstream parapet walls, sidewalks, curbing, crane rails, and control equipment. The cost for this preparatory work would be the same for the 6.5-, 12.5-, or 18.5-foot dam raises.

Modification of the main dam would require the demolition, removal, and transportation to waste of top-of-dam materials. This would include the demolition and removal of the upstream reinforced-concrete parapet wall and curb. Sawcuts would be used to aid in removing the upstream reinforced-concrete parapet wall and curb. In addition, sawcuts would be required along the upstream face and crest of the dam before the excavation of a 2-foot by 2-foot end area at the upstream face of the dam to embed a 12-inch polyvinyl chloride (PVC) waterstop. The existing dam crest would be prepared by using a high-pressure water jet on the concrete surface. The existing roadway drains would be backfilled with cement grout.

Equipment would be mobilized for drilling 4-inch-diameter drain holes on 10-foot centers from two different locations: from the existing dam crest to drain the surface contact, with each hole 2.5 feet long (248 holes), and from the existing dam crest for surface drainage at the downstream overhang, with one hole per block and each hole 6.5 feet long (50 holes). A 3-foot-diameter vertical shaft would be excavated through the concrete from the existing dam crest to the hoist gallery in Block 47 for installation of electrical conduit.

Concrete Dam and Wing Dams

Shasta Dam is located on the Sacramento River, approximately 9 miles northwest of Redding, California, and is a major feature of the CVP. As mentioned, the dam was designed and constructed by Reclamation, and was completed in 1945. The concrete dam portion has a structural height of 602 feet, a hydraulic height of approximately 480 feet (between reservoir water surface elevation 1,067 (NGVD29) and the original streambed at the axis of the dam), a crest width of 30 feet, and a crest length of 3,460 feet at crest elevation

1,077.5 (NGVD29). (Note that the following elevations presented in this section are based on the NGVD29 datum.) The downstream face of the concrete dam is vertical above elevation 1,050, with a slope of 0.8:1 below elevation 1,050, and the upstream face of the dam is vertical above elevation 720, with a slope of 0.5:1 below elevation 720. The concrete dam is slightly curved in plan view, with a large radius of 2,500 feet; however, the 375-foot-long spillway section located in the central portion of the concrete dam has a straight alignment. The volume of concrete in the main dam is 6,270,000 cubic yards. The dam impounds a reservoir with a total volume of 4,552,000 acre-feet at the top of joint-use storage, reservoir water surface (RWS) elevation 1,067. The focus of the feasibility study and this document is a dam raise of 18.5 feet. However, pertinent factors related to other proposed Shasta Dam raise heights of 6.5 and 12.5 feet have been considered for development of cost estimates.

The main dam would be raised 18.5 feet, from elevation 1,077.5 to elevation 1,096.0, to accommodate a 20.5-foot increase in the top of joint-use storage, from RWS elevation 1,067.0 to RWS elevation 1,087.5. The main dam raise would consist of mass and structural concrete placements for dam Blocks 15 through 38, and 46 through 71. The new dam crest would have the same surface area as the existing dam crest and similar features, including gantry crane rails and surface drains. A new upstream parapet wall would provide flood protection to elevation 1,099.5. The dam raise would include a new utility gallery, and 5-inch-diameter formed drains on 10-foot centers. Two rows of post-tensioned anchors spaced on 10-foot centers would be installed from the new dam crest to a depth of 92.5 feet within Blocks 30 through 38, and 46 through 50, for dynamic stability of the raised dam crest during a large earthquake. The existing elevator tow in Block 46 and the existing hoist tower in Block 35 would be raised to maintain their functions. Plate 4 shows typical sections of the concrete dam raise.

The mass concrete placements would use lift heights between 5 feet and 10 feet above the existing concrete surface at elevation 1,077.5, between contraction joints, and to the required 30-foot width. The contraction joints in the raised portion of the main dam would match the existing contraction joints, and would be keyed and grouted. Artificial cooling of the mass concrete placements would not be required because limits would be imposed on the placement temperature, and the concrete mix would be designed to limit the heat of hydration. The mass concrete would have a design compressive strength of 4,000 pounds per square inch (psi) at 365 days and would have 370 pounds of cementitious material per cubic yard of concrete, consisting of 50 percent pozzolan and 50 percent cement. Five-inch-diameter formed drains on ten-foot centers would be located in the center of the blocks from elevation 1,077.5 to the new dam crest at elevation 1,096.0, and would have caps for future inspection and maintenance.

Structural concrete would be placed for the top of dam above elevation 1,092.5, including concrete for the roadway, the upstream and downstream parapets, and the walkway. The structural concrete would have a design compressive strength of 4,000 psi at 28 days, would have 564 pounds of cementitious material per cubic yard of concrete, and would be made up of 20 percent pozzolan and 80 percent cement. Reinforcing bars would be used around the utility gallery, and nominal temperature steel would be used for the exposed structural concrete surfaces. Two 6-inch-diameter steel top-of-dam drains would be furnished and installed in each block to drain to the upstream face.

At each contraction joint, 12-inch PVC waterstops would be furnished and installed across the dam block contraction joints and around the utility gallery to provide a grout seal. Mobilization and demobilization would occur for pressure grouting the contraction joints. The 1.5-inch-diameter standard pipe metal tubing and fittings for the grouting system would be furnished and installed. There would be a total of 144 grout hookups based on 3 hookups at each of the 48 joints for the main dam. The system would be water-tested before pressure grouting. The final mix for the grout would use Type II cement and is assumed to have a water-cement ratio of 0.9:1, which requires 0.7 bags/cubic yard. Assuming 6 times the final volume to cover waste, the volume of grout per contraction joint is assumed to be 1 cubic yard.

Zoned embankment wing dams were originally constructed on both abutments of the main dam to protect the contact between the concrete and the excavated foundation surface. The tops of the embankment wing dams slope longitudinally from the main dam crest at the abutments down toward the spillway, approximately parallel to the excavated surface of the dam foundation, except the top of the right upstream embankment wing dam, which is approximately 200 feet below the crest of the main dam. The upstream face slopes at 2.5:1 and the downstream face slopes at 2.4:1 for both wing dams. The left wing dam includes a 450-foot-long concrete core wall beyond the left end of the concrete dam above elevation 980. The embankment wing dams contain approximately 2,160,000 cubic yards of earthfill materials.

The left wing dam would be raised 20.5 feet to elevation 1,098.0 to maintain the same height above the top of joint-use storage as for existing conditions. This would involve extending the existing reinforced-concrete core wall to the raised dam crest, and placing a thick layer of large rockfill downstream from the core wall to a slope of 2.5:1. The upstream face would consist of a reinforced concrete or mechanically stabilized earth (MSE) wall, and a concrete parapet wall to elevation 1,101.5. The road from the concrete dam crest would be ramped up through the left wing dam to the new embankment crest. Roadways and security features on the existing dam crest would be relocated to the new dam crest (see Plate 5). The existing rotunda on the left abutment of the dam would be removed and reconstructed.

The right wing dam would be raised 18.5 feet, from elevation 1,077.5 to elevation 1,096.0, which would involve extending the main dam raise from Block 71 to the right abutment gantry crane storage area (Block 77) using mass and structural concrete founded on bedrock. Concrete was selected for the right wing dam in lieu of embankment to facilitate construction. The new right wing dam crest would have the same surface area and similar features as for the existing right wing dam crest, including gantry crane rails and surface drains. A new upstream parapet wall would provide flood protection to elevation 1,099.5. The right wing dam would include a new utility gallery and a foundation drainage curtain (see Plate 6). The right abutment access roads would be modified to match the new dam crest, as shown in Plate 7. Construction quantities for the major items of work for these features are summarized in Table 3-1.

Table 3-1. Concrete Dam and Wing Dams Construction Quantities

Item	Main Concrete Dam Quantities	Right Wing Dam Quantities	Left Wing Dam Quantities
Concrete Removal (yd^3)	1,400	10	600
Concrete (yd^3)	56,700	8,750	2,850
Reinforcing Steel (lbs)	1,150,800	115,600	435,000
Crane Rails (lbs)	250,000	With main dam	N/A
Misc Metalwork (lbs)	65,400	With main dam	N/A
Post-Tension Anchors (lf)	13,440	N/A	N/A
MSE Wall (ft^2)	N/A	N/A	6,100
Embankment Core (yd^3)	N/A	N/A	9,100
Embankment Filter (yd^3)	N/A	N/A	3,900
Embankment Rockfill (yd^3)	N/A	N/A	85,000
Embankment Riprap (yd^3)	N/A	N/A	8,000

Key:
ft^2 = square feet
lbs = pounds
lf = linear feet
Misc = miscellaneous
MSE = mechanically stabilized earth
N/A = not applicable
yd^3 = cubic yards

More details regarding the concrete dam and wing dam raise designs are contained in Reclamation *Technical Memorandum No. SHA-8110-FEAS-2007-1* (2007d), *No. SHA-86-68110-FEAS-2008-1* (2008f), *No. SHA-86-68110-CD-2011-1* (2011a), and in Reclamation's *Left Wing Dam Raise Feasibility – Design Report* (2009a).

Spillway

Spillway releases are controlled by three 110-foot-wide by 28-foot-high steel drum gates located within the concrete overflow (spillway) section of the dam. The drum gates are hinged and anchored along the upstream side to a reinforced-concrete cantilever wall section. Rubber seals are located on the ends and downstream lip of each drum gate to form a watertight seal, allowing regulation of the spillway gate heights by adjusting the water levels inside the float chambers. The total discharge capacity of the existing spillway is 186,000 cubic feet per second (cfs) at RWS elevation 1,065 (NGVD29). (Note that the following elevations presented in this section are based on the NGVD29 datum). Two-foot-high steel flashboards operated from a walkway beneath the spillway bridge allow for reservoir storage between the top of the raised drum gates at elevation 1,065 and the top of joint-use storage at elevation 1,067.

Structural concrete would be used to raise the existing spillway crest from elevation 1,037 to elevation 1,049.5, and to shape the raised spillway crest (see Plate 8). The existing spillway bridge, the two existing spillway piers, the cantilever wall sections, and the three existing drum gates and operating equipment would be removed. Five new spillway piers would be constructed at locations within the spillway designed to avoid existing overflow block contraction joints, and a new concrete spillway crest would be constructed between the piers. One row of post-tensioned anchors variably spaced (average 10-foot centers) would be installed from the top of each spillway pier to a depth of 100 feet for dynamic stability during a large earthquake. The locations of the new piers would result in different widths of spillway gates. The three existing 110- by 28-foot drum gates would be replaced with six sloping, fixed-wheel gates (four 48- by 38-foot and two 54- by 38-foot gates). The total spillway crest length would be reduced from 330 feet to 300 feet as a result. Sloping, fixed-wheel gates were selected in lieu of radial gates to reduce potential seismic loads on the spillway piers during a large earthquake. Cross-bracing would be installed at the tops of the spillway piers to serve as gate seats in the raised position, and to reduce the height of the gates. Stop log guides would be placed immediately upstream from the spillway crest for gate maintenance. Two sets of stop logs (48 and 54 feet long) would be provided to service the six gate openings.

Additionally, an aeration system would be constructed in the spillway chute at approximately elevation 875 to mitigate potential cavitation damage during spillway releases, as shown in Plate 9. The total discharge capacity of the raised spillway is estimated to be 266,300 cfs at elevation 1,087.5.

A new bridge would be required to span the spillway to allow for vehicular traffic and for a gantry crane to travel from the right end of the dam to the far end of the spillway (see Plates 10 and 11). The existing spillway bridge consists of three 100-foot spans, while the new bridge would consist of six shorter spans. The spans would not be equal, because the pier locations were set to avoid the existing dam contraction joints. The spillway bridge is designed to be continuous over the pier supports; therefore, there are no deck expansion joint details except at the abutment ends of the bridge. Construction quantities for major items of work for this feature are summarized in Table 3-2.
Figure 3-1 is an artist rendering of the 18.5-foot dam raise option.

Table 3-2. Spillway Construction Quantities

Item	Quantity
Concrete Removal (yd^3)	6,850
Removal of 3 Drum Gates (lbs)	3,000,000
Removal of Bridge Superstructure (lbs)	755,000
Concrete (yd^3)	29,500
Reinforcing Steel (lbs)	1,980,000
Post-Tension Anchors (lf)	8,400
Sloping Fixed Wheel Gates (lbs)	3,315,600
Stoplogs and Guides (lbs)	753,000

Key:
lbs = pounds
lf = linear feet
yd^3 = cubic yards

Figure 3-1. Shasta Dam 18.5-Foot Raise

More details regarding the spillway structural and hydraulic designs are contained in Reclamation *Technical Memorandum No. SHA-8110-FEAS-2007-1* (2007d), *No. SHA-86-68110-FEAS-2008-1* (2008f), and *No. SHA-8130-FEAS-2011-1* (2011d).

River Outlets

The outlet works consist of eighteen trash-racked, 102-inch-diameter steel-lined conduits that pass through the concrete overflow (spillway) section – six at elevation 942, eight at elevation 842, and four at elevation 742 (NGVD29). Outlet releases are controlled by 102-inch-diameter tube valves for the four lower tier outlets, and by 96-inch-diameter wheel-type outlet gates for the fourteen middle tier and upper tier outlets. The river outlet works conduits, upstream from the regulating tube valves and outlet gates, can be closed by installing a portable coaster gate using the 125-ton gantry crane located on the crest of the dam. The single portable coaster gate is 11.05 feet square, and is stored within a structure at the right abutment of the dam. Total discharge capacity of the outlet works is 81,800 cfs at RWS elevation 1,067 (NGVD29).

Because of existing operational limitations associated with the four lower tier 102-inch-diameter tube valves, these valves would be replaced by four 96-inch-diameter jet flow gates, as shown in Plate 12. A downstream air shroud and vent would be provided for each gate, which would discharge into the existing 102-inch-diameter conduits. Construction quantities for the major items of work for this feature are summarized in Table 3-3.

Table 3-3. River Outlets Modifications Construction Quantities

Item	Quantity
Concrete Removal (yd^3)	350
Concrete (yd^3)	300
Reinforcing Steel (lbs)	30,000
Four 96-Inch Jet-Flow Gates (lbs)	375,400
Coaster Gate Guide Extensions (lbs)	178,200

Key:
lbs = pounds
yd^3 = cubic yards

Power Outlets

The Shasta Powerplant contains five main generating units and two station service units fed by five 15-foot-diameter penstocks that pass through the concrete dam to the right of the spillway section (intake centerline at elevation 815 (NGVD)). The steel penstock pipes extend over 500 feet from the downstream face of the dam to the powerplant structure, and are supported by concrete saddles. Each penstock pipe has a discharge capacity of 2,800 cfs and an upstream 15-foot by 19.05-foot coaster gate for emergency closure. The total discharge capacity of the powerplant is 14,000 cfs under a gross head of 480 feet. The maximum total generating capacity of the main units is 710 MW. Two of the five penstock pipes were modified in 1998 to supply water to a downstream fish hatchery located on the right bank of the downstream channel.

The existing steel penstock pipes have been determined to be adequate for the higher reservoir loads. Some modifications are anticipated for the power intake gate hoists to accommodate the higher dam crest. Although vulnerable to seismic loads, penstock failure due to an earthquake is not considered to be a dam safety issue, and modifications to the existing penstocks and concrete saddle supports are not proposed at this time. However, higher reservoir levels associated with the proposed dam raise would increase the normal head on the penstocks and associated damage in the event of a large earthquake. Construction quantities for the major items of work for this feature are summarized in Table 3-4.

Table 3-4. Power Outlets Construction Quantities

Item	Quantity
Relocate Hydraulic Hoist Systems (lbs)	105,000
Gate Stem Extensions (lbs)	12,500
Coaster Gate Guide Extensions (lbs)	15,000

Key:
lbs = pounds

Temperature Control Device

Construction of the Shasta TCD to allow selective withdrawal of the cooler reservoir water for discharge through the five powerplant penstocks was completed in 1996 to enhance the downstream fishery. The TCD is a steel structure consisting of a shutter structure and a low-level intake structure that are attached to the upstream face of the dam. Plate 13 shows a plan view of the TCD, with elevations and sections shown in Plates 14, 15, and 16. Conservation of the cold-water pool is achieved by forcing withdrawal from the highest elevation possible. To that end, the upper shutter gates, followed by the middle shutter gates, followed by the pressure relief gates, are operated to access the highest permissible level of withdrawal based on the RWS elevation and downstream water quality objectives. The design flow through the TCD is 19,500 cfs. Major TCD modifications necessary for the 18.5-foot dam raise include the following:

- Disassemble, reinstall, and modify gate hoists to accommodate the longer ropes required.
- Remove and reinstall motor control centers and distribution switchboards.
- Remove and reinstall hoist platforms on new rigid frame box girders to elevate the gate hoist and electrical equipment above the raised maximum RWS elevation.

- Install new dam connections for new rigid frames.
- Remove upper segments of trash racks and replace them with barrier panels on the upstream face of the TCD with cladding panels along the sides of Shutters 1 and 5 to reduce undesirable mixing of reservoir water during controlled downstream releases.
- Extend gate guides, and cladding guides, barrier panels, cladding panels, and closure panels.
- Remove and reinstall miscellaneous metalwork (e.g., grating, pipe guardrails) and provide new miscellaneous metalwork (e.g., grating, pipe guardrails, platforms, ladders, safety cages) to account for the dam raise, modified platform member configurations, and safety requirements.
- Attach a new debris boom to existing lake and dam anchorages to exclude debris from the TCD and spillway (see Plate 17), and provide equipment to remove and transport debris from the lake.

Construction quantities for the major items of work for this feature are summarized in Table 3-5.

Table 3-5. Temperature Control Device Modifications Construction Quantities

Item	Quantity
Steel Removal and Disposal (lbs)	253,000
Remove and Reinstall Hoist Platform Steel (lbs)	740,000
Remove and Reinstall Misc Metalwork (lbs)	150,000
Remove and Reinstall Hoists (each)	17
Rigid Frame Steel (lbs)	562,000
Hoist Platform Steel (lbs)	50,000
Misc Metalwork (lbs)	31,000
Cladding Panels and Guides (lbs)	200,000
Front Gate Guides (lbs)	334,000
Barrier Panels (lbs)	465,000
Debris Boom (each)	1

Key:
lbs = pounds

More details regarding the TCD designs are contained in the Reclamation *Technical Memorandum No. SHA-8120-FEAS-2007-1* (2008g).

Visitor Center

The existing visitor center building is located on the left abutment of Shasta Dam, as shown in Plate 3, and provides office space for Reclamation's Northern California Area Office (NCAO) in addition to visitor space, storage areas, and visitor center staff offices. With the dam raise, visitors would have to be conveyed approximately 21 vertical feet to access the new dam crest. The current access point to the parking lot would not be usable because of the vertical realignment of Shasta Dam Boulevard and the construction of a concrete retaining wall associated with the dam raise. The existing security office is housed in an adjacent building that would be too low with respect to the new dam crest elevation for effective observation of the site facilities.

As a part of the overall Shasta Dam raise feasibility design, preliminary designs were prepared for a new visitor's facility with a security office and associated site improvements. It is assumed that the existing building would be remodeled for sole occupancy by the NCAO and Shasta Dam administration staff. The remodel of the existing building is not currently included in the dam raise project.

The proposed 11,000-square-foot visitor center building would provide adequate space for visitors, storage, staff, and security functions, and feature a panoramic view of all facilities (see Plate 18). A modern theater would have state-of-the-art media equipment and a 14-foot-wide suspended screen. An outdoor, terraced lawn area would be integrated into the site design for picnics and viewing. A security checkpoint would be located at the beginning of the powerplant access road. An elevator, stairs, and a covered pedestrian bridge would provide access to the new dam crest. The new design would comply with all accessibility, life safety, and seismic safety standards, and would demonstrate a commitment to sustainable building design and alternative energy uses. The existing parking lot would be redesigned to provide efficient, safe, and secure vehicular and pedestrian circulation through the site for visitors, NCAO staff, security, and maintenance operations. Figures 3-2 and 3-3 are artist renderings of the proposed visitor center.

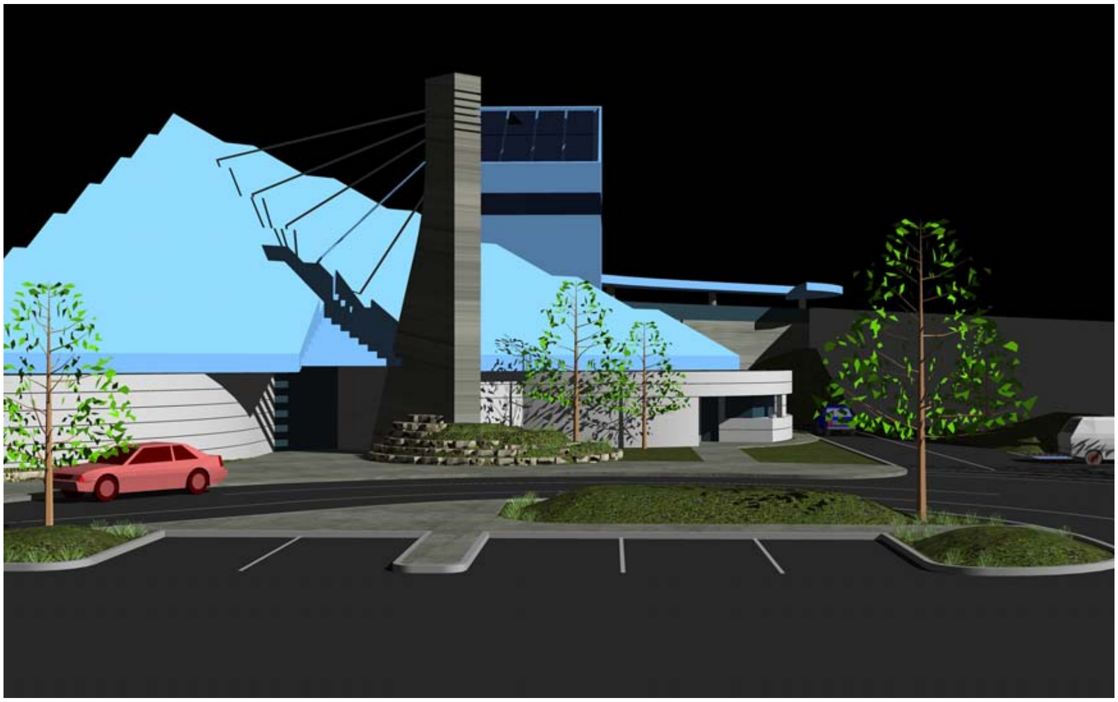

Figure 3-2. Visitor Center Entrance

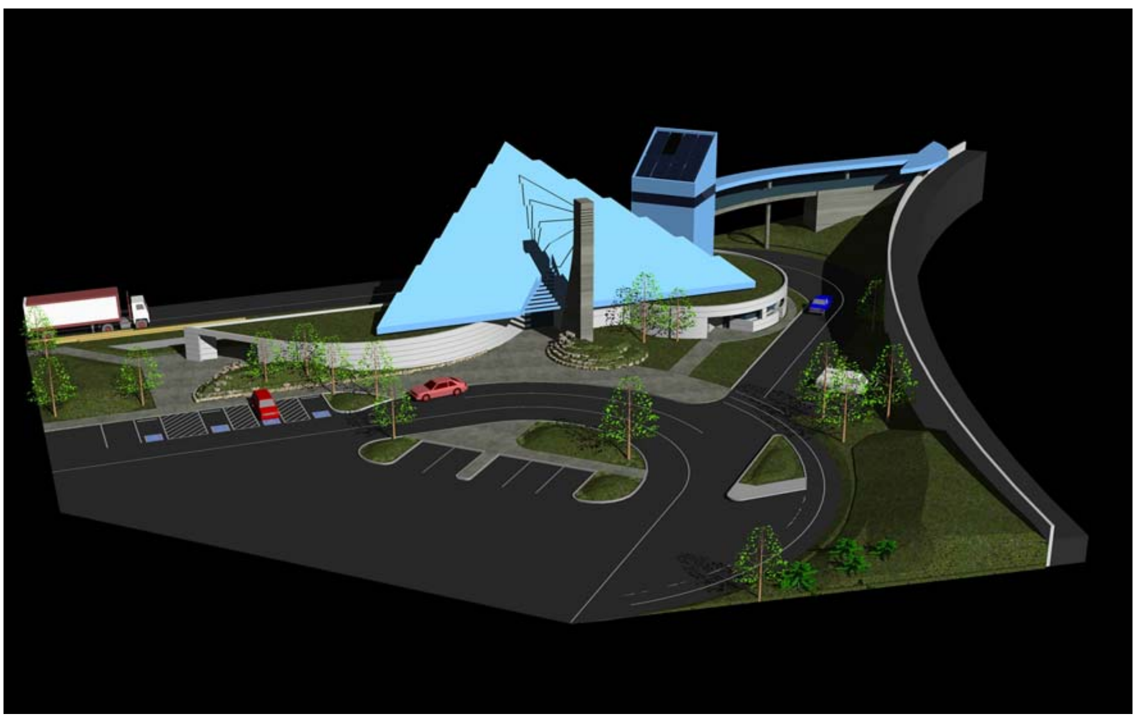

Figure 3-3. Visitor Center and Surrounding Area

The proposed architectural design would be visually compatible with the dam and with other outstanding regional features. A freestanding vertical, reinforced concrete pier structure with an array of tension rods would support the central beam of a triangular metal roof. The vaulted triangular roof would be stepped with north-facing vertical clear-story windows to provide an open and unimpeded view of the dam. Curved exterior concrete walls with earthen embankments would help moderate heat gain and loss. The ground floor security offices on the east side and visitor center staff offices on the west side would feature vegetated roofs of native grass species. An adjoining four-story metal building would contain a stairwell, elevator, and access to the theater, pedestrian bridge, and fourth floor security observation room. Construction quantities for the major items of work for this feature are summarized in Table 3-6.

Table 3-6. Visitor Center Construction Quantities

Item	Quantity
Site Excavation (yd^3)	9,400
Compacted Backfill (yd^3)	4,250
Topsoil (yd^3)	670
Asphalt Pavement (yd^3)	1,100
Solar Panels (ft^2)	11,800
Glass Curtain Wall (ft^2)	4,000
Exterior Metal Siding (ft^2)	5,100
Metal Roofing (ft^2)	13,545
Concrete (yd^3)	820
Reinforcing Steel (lbs)	115,000

Key:
ft^2 = square feet
lbs = pounds
yd^3 = cubic yards

Lands

The Shasta Lake area lands valuation accounts for cost of acquiring public and private land required for the project because of new inundation, and permanent and temporary construction easements for reservoir area facility relocations. For the 6.5-foot, 12.5-foot, and 18.5-foot dam raise options, other than in the vicinity of Lakeshore, relatively few additional non-Federal acres of land would need to be acquired through easement or purchase. Table 3-7 shows the affected parcels by dam raise option. The Real Estate Appendix provides estimated land requirements and estimated cost.

Table 3-7. Parcels Affected by Dam Raise Options

	CP1 (6.5-Foot)	CP2 (12.5-Foot)	CP3 (18.5-Foot)	CP4 (18.5-Foot)	CP5 (18.5-Foot)
Total Non-Federal Parcels Affected	80	131	179	202	202
USFS Land Cabins Affected	24	27	28	28	28

Key:
CP = Comprehensive Plan
USFS = U.S. Forest Service

Clearing of Reservoir Area

An increase in the height of Shasta Dam and Reservoir is expected to inundate varying areas of vegetated shoreline and riparian areas, depending on the final dam raise height. Inundated areas are described in Table 3-8; corresponding vegetation management is discussed in a later section. Inundation levels currently vary by water year and season at Shasta Lake, and this variation would continue if the dam were raised. Maximum water surface occurs during late spring, consistent with flood control operating guidelines. The reservoir is drawn down steadily throughout the summer in response to water supply demands. Based on this historic pattern, the new inundation zone would be exposed during most of the year for most water year types. Treatment windows and access are very limited by water levels and public use of the reservoir. Thus, a thorough vegetation removal strategy would be necessary to facilitate the continued safe operation of the dam and recreation area.

Table 3-8. Inundated Areas and Corresponding Vegetation Prescriptions

Description		Vegetation Removal Prescription		
Dam Raise (feet)	Total Inundated Area (acres)	No Treatment (acres)	Overstory Removal (acres)	Total Removal (acres)
6.5	1,110	740	220	150
12.5	1,750	1,167	347	236
18.5	2,500	1,668	495	337

Rationale and Need for Vegetation Management

The majority of the area now occupied by Shasta Lake was completely cleared of vegetation during construction of the original dam but some vegetation was left in place. The Pit River arm of the reservoir represents an area that was not cleared because of a shortage of available workers and resources. This area is an example of the effects of not clearing vegetation. Many snags are still standing in the Pit River arm, providing unique wildlife habitat. If left untreated, inundated upland vegetation is not expected to survive the first period of sustained inundation. Riparian vegetation would survive most of the expected inundation periods, although occasional mortality would be experienced over the life of the project.

The goals of preconstruction vegetation management are as follows:

- Reduce hazards to the public using the lake for recreation.
- Provide access to the shoreline near high-use areas.
- Maximize retention of habitat components that would survive inundation.
- Minimize impacts to operation of the dam and spillway.

If left in place, inundated vegetation could pose a hazard to the recreating public and physically block access to the newly created shoreline. Snags or fallen trees could pose a navigation hazard to boaters.

Brush, particularly manzanita, provides effective rearing cover for fish when inundated. Manzanita would be removed in clearing areas and stockpiled to be used for fish habitat structures placed in designated locations; however, willows, cottonwoods, and buttonbush would not be removed in and along riparian areas. Dead upland vegetation would provide some habitat during periods of inundation. Snags caused by inundation would provide habitat for birds and cavity nesters. During periods of reservoir drawdown, the residual root systems of inundated vegetation would provide support to newly created shorelines, reducing erosion. The benefits provided by dead vegetation would decrease over time as roots, plant skeletons, and snags decayed.

To minimize operational hazards from small debris loading after vegetation removal, booms and other methods would be employed to capture debris, and are intended to provide better protection than currently exists for the dam and TCD. Plate 17 shows the plan, sections, and details for a proposed reservoir boom.

Proposed Vegetation Management Prescriptions

Three vegetation removal prescriptions are proposed for the areas around the reservoir that would be inundated, as described in the following sections.

Complete Vegetation Removal

Complete vegetation removal treatment would clear all existing vegetation from a designated treatment area and would generally be applied to areas along and adjacent to developed recreation areas, including boat ramps, day use areas, campgrounds, marinas, and resorts. Plate 19 shows the various vegetation management prescriptions around the reservoir. Exceptions would be made in areas with high shoreline erosion potential or in habitat for special-status species.

Timber would be harvested and removed to landings by ground skidding equipment if road access were present and slopes were less than 35 percent; otherwise, trees would be yarded by helicopter, and residual vegetation and activity-created slash would be piled and burned by hand. Where possible, trees would be felled into the reservoir during removal to minimize damage to reservoir walls. Tree stumps would be cut to within 24 inches of the ground surface and brush stumps would be cut flush to the ground. Stumps would be left in place to reduce shoreline erosion. This treatment is intended to maximize shoreline access and minimize the risk to visitors from snags and water hazards.

Overstory Removal

Overstory removal treatment would remove all trees greater than 10 inches in diameter at breast height (DBH) or 15 feet in height from the treatment area, and would generally be applied to houseboat mooring areas or narrow arms of the reservoir where snags would pose the greatest risk to boaters. Trees would be harvested and removed to landings by ground skidding equipment if road access were present and slopes were less than 35 percent; otherwise, trees would be yarded by helicopter, and activity-created slash would be piled and burned by hand. The remaining understory vegetation would be left in place. Where possible, trees would be felled into the reservoir during removal to minimize damage to reservoir walls. Tree stumps would be cut to within 24 inches of the ground surface. Stumps would be left in place to reduce shoreline erosion. This treatment is intended to minimize the risk to visitors from snags and water hazards.

No Treatment

Designated areas of the inundation zone would be left untreated and no vegetation would be removed. This prescription would generally be applied to stream inlets, the upper end of major drainages, and the shoreline of wider arms of the reservoir. This prescription would also apply to special habitat areas, and is intended to maximize the habitat benefits of inundated and residual vegetation.

Vegetation Management Areas

Fifteen vegetation management areas have been delineated to facilitate the efficient removal of vegetation around the reservoir perimeter. The acreages of each management prescription are summarized in Table 3-9 by vegetation management area.

Table 3-9. Vegetation Management Prescription Summary by Area

Landing	Proposed Dam Raise											
	6.5 feet				12.5 feet				18.5 feet			
	Overstory Removal (acres)	Overstory Removal Quantity (board feet)	Total Removal (acres)	Total Removal Quantity (board feet)	Overstory Removal (acres)	Overstory Removal Quantity (board feet)	Total Removal (acres)	Total Removal Quantity (board feet)	Overstory Removal (acres)	Overstory Removal Quantity (board feet)	Total Removal (acres)	Total Removal Quantity (board feet)
Antlers	5	33,400	8	48,600	8	52,700	12	76,600	12	75,100	17	109,300
Bailey Cove	7	40,600	17	148,400	11	64,000	26	234,000	15	91,300	37	333,700
Beehive Point	24	102,300	3	5,400	38	161,300	4	8,500	54	230,100	6	12,100
Bridge Bay	0	0	9	51,800	0	0	14	81,600	0	0	20	116,400
Digger Bay	31	92,600	8	27,700	49	146,000	13	43,700	70	208,300	19	62,400
Hirz Bay	22	169,500	22	211,200	34	267,300	35	333,000	49	381,200	49	474,900
Jones Valley	51	328,000	17	81,700	81	517,100	26	128,800	116	737,500	38	183,700
Lakeshore East	2	12,500	17	58,800	4	19,700	27	92,800	5	28,100	39	132,300
Lower Salt Creek	15	62,700	14	96,300	24	98,900	22	151,800	35	141,100	31	216,500
McCloud Arm	0	0	4	14,900	0	0	7	23,500	0	0	10	33,500
Packers Bay	22	78,800	7	29,200	35	124,200	11	46,000	50	177,100	16	65,600
Pit Arm	0	0	2	22,400	0	0	3	35,300	0	0	4	50,400
Shasta Marina	13	89,400	1	17,900	21	141,000	2	28,200	30	201,100	2	40,200
Silverthorn	18	115,100	17	117,900	29	181,400	26	185,900	41	258,800	37	265,200
Turntable	8	88,700	5	33,100	13	139,900	8	52,200	19	199,500	11	74,400
Total	220	1,213,600	150	965,300	347	1,913,500	236	1,521,900	495	2,729,200	337	2,170,600

A single staging area (landing) would serve each vegetation management area. Access for vegetation removal activities would most likely be limited to late summer and fall, when water levels were low and recreation use had decreased. Removal by helicopter would generally be limited to spring and fall because of the limited availability of helicopters during the summer fire season. Vegetation removal would also be limited during bird nesting season, typically February through September. Reservoir area breeding surveys would be performed before vegetation removal activities in an effort to avoid nesting species.

The average distance for helicopter trips per vegetation management area is described in Table 3-10. Note that because of distance and/or safety constraints, helicopters would not be used in the following vegetation management areas: Bridge Bay, Lakeshore East, Pit Arm, and McCloud Arm. Slash burning could take place during the winter following vegetation treatment, and would comply with all regulations set forth by the Shasta County Air Quality Management District. Vegetation management activities would need to be complete before inundation of new areas created by a dam raise.

Table 3-10. Average Flight Distance for Each Vegetation Management Area

Landings	Average Distance per Flight (miles)
Antlers	0.8
Bailey Cove	1.3
Beehive Point	1.2
Digger Bay	2.5
Hirz Bay	1.5
Jones Valley	2.7
Lower Salt Creek	1.1
Packers Bay	1.7
Shasta Marina	1.3
Silverthorn	1.0
Turntable	1.1

Reservoir Area Dikes

With Shasta Dam enlargement scenarios, dikes in the Lakeshore and Bridge Bay areas would be required to protect California Department of Transportation (Caltrans) highways, the Union Pacific Railroad (UPRR), and other infrastructure from inundation. The focus of the feasibility study and this document is a raise of 18.5 feet. However, pertinent factors related to other proposed Shasta Dam raise heights of 6.5 and 12.5 feet are considered.

Two closure dikes and three railroad embankments in the Lakeshore area would be required in support of an 18.5-foot dam raise. Locations of the proposed dikes are shown in Plates 20 and 21 for each of the proposed dam raise alternatives. Table 3-11 summarizes the type of construction proposed for each of the dikes, including total volumes of each type of material required. The dikes are named to correspond with those presented in the *Plan Formulation Report* (PFR) (Reclamation 2007e). Some of the dikes originally proposed in the PFR were not required for feasibility analysis because of the new proposed alignments of the dikes. Doney Creek and Antlers Dikes were originally named Dikes 8 and 10, respectively, in the PFR.

Table 3-11. Estimated Fill Volumes Required for Proposed Lakeshore and Bridge Bay Dikes (18.5-foot dam raise)

Proposed Dike	Dike Construction Type	Core (cy)	Drain (cy)	Filter (cy)	Riprap (cy)
Lakeshore Dikes					
Doney Creek Dike	Homogenous	56,300	1,330	3,040	5,920
Antlers Dike	Homogenous	3,700	70	190	760
North RR Dike	Homogenous	13,100	-	690	410
Middle RR Dike	Homogenous	10,200	-	540	320
South RR Dike	Homogenous	77,900	-	4,110	2,460
Subtotal		161,200	1,400	8,570	9,870
Bridge Bay Dikes					
West Dike	Homogenous	35,100	630	21,880	23,630
East Dike	Homogenous	25,600	310	6,950	7,440
Subtotal		60,700	940	28,830	31,070
Total[1]		**221,900**	**2,340**	**37,400**	**40,940**

Note:
[1]Volumes exclude swelling factors

Key:
- = not applicable
cy = cubic yard
RR = railroad

Typical cross sections for homogenous fill dikes for an 18.5-foot dam raise are shown in Plate 22. Cross sections would be similar for 6.5- and 12.5-foot raises. Plate 23 shows typical cross sections of railroad embankments. It is expected that approximately 3–5 feet of organic-rich soil and vegetation would be excavated from the foundation of the dike, and from a shear key on the upstream side of the dike. Riprap would be placed on the upstream face of each dike to the crest of the dike to provide protection from wave run-up and erosion.

For the purposes of this feasibility study, *Reclamation Design of Small Dams* (Reclamation 1987) guidelines were used to generate typical dike cross sections. Homogenous fill dikes would consist of relatively impervious fill with embankment slopes of 3:1 upstream and 2.5:1 downstream, as shown in Plate 22. A low-permeability core would extend to the crest of the dike and have upstream and downstream slopes of 2:1. Subsequent phases of geotechnical investigation during final design may lead to revisions of these typical cross sections.

Doney Creek Dike

For the proposed 18.5-foot alternative, the Doney Creek Dike would extend approximately 1,800 feet along the southeastern side of the UPRR embankment to the north of the Doney Creek Bridge, turning to the west underneath the Doney Creek Bridge along the shoreline of Doney Creek, as shown in Plate 20. The purpose of the Doney Creek Dike would be to protect the UPRR embankment to the north of the Doney Creek Bridge from partial inundation when water reached ordinary high water levels as a result of the 18.5-foot dam raise alternatives.

Dike construction would consist primarily of homogenous fill in the flat areas to the south of the existing UPRR embankment. The use of homogenous earth dikes in this area is intended to limit construction costs. A combination of flood walls and zone embankments would be required in areas where construction access would be limited or where natural slopes exceeded the slopes of the embankments. The limited overhead construction space below the Doney Creek Bridge would make constructing an earthen dike infeasible, since the upper lifts of the dike would not be accessible with a compactor. Accordingly, a flood wall would be required at this location. Similarly, the natural slopes of the ravine located north of the Doney Creek Bridge are steeper than the anticipated slopes of the homogenous dikes. A rock fill dike or a dike supporting a floodwall would be required at this location.

Antlers Dike

For the proposed 18.5-foot alternative, the Antlers Dike would extend approximately 200 feet in a northeasterly direction away from Interstate 5. The purpose of the Antlers Dike would be to protect Interstate 5 from partial inundation when water reached ordinary high water levels for the 18.5-foot dam raise. Dike construction would consist of a homogenous fill embankment, which would be the most cost-effective and appropriate for this small dike.

Railroad Embankments

Three railroad embankments, labeled North, South, and Central (see Plate 24), are proposed along the railroad alignment in the Lakeshore area, located between the Doney and Charlie creek arms of Shasta Lake. The purpose of these embankments would be to support the railroad for the adjusted alignment. The North Embankment would extend approximately 1,700 feet from the north shore of Charlie Creek in a northern direction. The South Embankment would be to the north of the first embankment and would extend approximately 1,100 feet along the proposed railroad alignment. The Central Embankment would extend approximately 340 feet along the proposed railroad alignment, terminating at the south shore of Doney Creek.

Bridge Bay West and East Dikes

At the Bridge Bay Marina, a 700-foot long section of UPRR tracks stretches between two railroad tunnels, located downslope from Interstate 5. For the proposed 18.5-foot dam raise alternative, this section of railroad tracks would be inundated as a result of ordinary high water. Dikes are proposed on either side of the tracks to protect the railroad from inundation both by Shasta Lake and impounded stormwater runoff from the hillside directly east of the tracks, as shown in Plate 25.

Geotechnical analysis of the existing foundation material for the Bridge Bay dikes determined that significant excavation would be required for the west Bridge Bay Dike. To minimize the dike footprint and limit excavation, jet grouting was proposed. The existing unsuitable foundation material for the east Bridge Bay Dike was determined to extend to a maximum depth of 15 feet, and is reflected in the 18.5-foot dam raise estimates and figure.

Borrow Material for Dikes

The Lakeshore and Bridge Bay dikes would be constructed as homogenous embankments. Homogenous dikes would consist primarily of low-permeability fill with a small granular drain and filter at the toe of the embankment. Dike slopes retaining water would be armored with riprap. An overview of the borrow material needed for construction of the proposed dikes and description of potential borrow sources is provided in the following paragraphs.

Borrow Material Classification

Dike designs consider embankment fill materials, including homogenous fill, core, riprap, filter, and drain materials. Brief descriptions of these materials are given below.

Core and Homogenous Fill Material Feasibility designs of dikes constructed as homogenous embankments would be constructed almost entirely of low-permeability soil. Typically, core fill materials consist of impervious fine-grain materials, such as silts and clays, or coarse-grain materials, such as gravels or sands with significant components of clay or silt.

Riprap Riprap is used to protect embankment slopes subjected to wave erosion. Properly graded riprap is commonly used to provide slope protection. Riprap needs to be placed in a manner to provide a well-integrated mass with minimum void spaces. Generally, riprap consists of a uniform distribution of angular and durable gravel- to boulder-sized rock.

Filter Material Granular filters will be required for each of the proposed embankments in the Lakeshore and Bridge Bay areas. Granular filters are used to minimize the risk of internal erosion at the boundaries of dissimilar fill materials. The filters consist of uniformly graded, free-draining materials with less than 5 percent fines by weight. Granular filters are restricted to a maximum particle size of 3 inches to avoid segregation and bridging of large particles during construction.

Drain Material The proposed homogenous fill embankments would be constructed with toe drains to minimize pore pressures within the embankments. Toe drains can be used in concert with blanket drains to further reduce uplift pressures along embankment foundations. Toe drains must be strong, durable, and free draining. Typically, drain materials consist of sands and gravels with less than 5 percent fine-grained soil.

Borrow Requirements for Dikes

Assuming a crest elevation of 1,098.7 feet (NAVD88) for the Doney Creek and Antlers dikes, and 1,104 feet (NAVD88) for Bridge Bay East and West dikes, estimates of the borrow materials needed to construct the proposed dikes are presented in Table 3-11. The proposed dike geometries were developed for feasibility evaluations. Accordingly, volumes of core and homogenous fill, riprap, filter, and drain materials are estimates, and may be refined during subsequent phases of design. The required fill volumes are presented in greater detail in the SLWRI *Reservoir Area Dikes and Related Facilities Report* (Reclamation 2011b).

Other Considerations

The following sections discuss other dam raise options considerations relating to Pit 7 Dam, Keswick Dam, probable maximum flood (PMF), borrow sources, and geological and geotechnical information.

Pit 7 Dam and Powerhouse

This section presents design and construction information to support the feasibility assessment of the Pit 7 Dam, Powerhouse, and related facilities associated with the proposed raise of Shasta Dam. The Pit 7 Dam and Powerhouse, which is owned and operated by Pacific Gas and Electric (PG&E), is located on the upper Pit River at the northeast end of Shasta Lake. The complex consists of three main features: a main dam with integral spillway, a two-unit hydroelectric powerhouse immediately downstream from the main dam, and an afterbay dam (see Plate 24). Each main feature, and the impacts of the three Shasta Dam and Reservoir raise options on each feature, are discussed below. Costs are included in the estimates for potential physical impacts to the Pit 7 Powerhouse. No modifications are identified or included for the main dam or afterbay facilities.

The main Pit 7 gravity dam was evaluated to determine the impact of the increased tailwater level caused by a Shasta Dam raise on the uplift pressures under the base of the dam. Normal operating conditions for the three Shasta

Dam raise alternatives (6.5, 12.5, and 18.5 feet) were considered in the stability analysis. An extreme loading combination, including seismic loads, was also considered in the analyses. For this evaluation, the computer program CADAM 2000 (Version 1.4.11) was used to calculate factors of safety against sliding and overturning of the dam. Compressive and tensile stresses were also investigated at the heel and toe of the dam. Stability analysis results can be found in the SLWRI *Pit 7 Dam and Powerhouse Facilities Report* (Reclamation 2008).

Pit 7 Spillway

The spillway was evaluated to determine if a potential modification or raising of the Pit 7 Dam spillway flip bucket would be required because of higher tailwater levels caused by a Shasta Lake raise and concurrent Pit 7 Dam spillway flows. Tailwater levels above the lip of the spillway combined with substantial spillway flows could potentially affect spillway performance and have adverse effects on the dam and adjacent powerhouse.

Backwater conditions above the lip of the spillway could potentially interfere with spillway performance. If the spillway discharge was small enough or if the tailwater was high enough, sweep-out of the flip bucket would not occur and the roughly 85-foot-long flip bucket would function in a manner similar to a stilling basin. If the spillway flows were great enough to cause sweep-out of the flip bucket, a high tailwater level could interfere with the intended trajectory of the spill and cause energy dissipation and potential scour to occur near the terminus of the spillway. This is a condition that should be avoided. At all tailwater levels below the lip of the flip bucket, the flip bucket would either function as a stilling basin for low flows, or when the spillway flow was high enough, the spill would be thrown a distance downstream and energy dissipation would occur at an acceptable distance away from the dam.

Conditions that could cause potential energy dissipation problems would require tailwater caused by backwater above elevation 1,075.5 (NGVD29) coincident with high spillway flows.

The 50-year historical and simulated record does not indicate any spillway discharge events that would likely be damaging because of the potential Shasta Dam raises. Thus, it can be projected that the Pit 7 Dam should not be subjected to damaging flows for the potential dam raise alternatives up to the 1 percent chance of occurrence each year. This is not an indication that damaging conditions at the spillway would not occur for all flood events up to the PMF, but the damage potential from overtopping of the Pit 7 Dam would probably exceed that which could be attributed to spillway performance alone. Therefore, it is recommended that there should be no requirement to raise the Pit 7 Dam spillway as a direct result of the Shasta Dam raise options that are under consideration.

Pit 7 Powerhouse

The powerhouse is a semi-outdoor type powerhouse, with two generating units each rated at 56 MW output under approximately 204 feet of net head. The main powerhouse yard/deck, where the generators and generator step-up transformer are located, is at elevation 1,104.25 feet (NGVD29). (Note that the following elevations presented in this section are based on the NGVD29 datum). A training wall with top at elevation 1,094 feet separates the dam spillway from the powerhouse. The normal tailwater level under existing conditions is elevation 1,067 feet. A raise of Shasta Dam by 18.5 feet would raise the normal tailwater to elevation 1,087.5 feet. This would still provide 6.5 feet of freeboard to the top of the training wall and 16.75 feet of freeboard to the powerhouse yard/deck. The overall powerhouse would not be inundated, but other effects need to be considered/addressed.

The Pit 7 Powerhouse is sometimes operated in synchronous or “motoring” mode to balance the power grid if demand drops below available generation. In this condition, the water in the draft tube is lowered to an elevation below the runner while the unit is at synchronous speed. A set of breakers is closed and surplus power from the grid is used to turn the generator similar to a motor without water flowing through the turbine. At the Pit 7 Powerhouse, the normal tailwater is below the bottom of the runners. This enables the Pit 7 Powerhouse to switch to synchronous mode by simply closing the wicket gates and allowing the tailwater to equalize in the draft tubes at the current stream elevation, then closing the breakers to switch to “motoring” mode. With an increased tailwater elevation, it would be necessary to install a tailwater depression system to lower the water level in the draft tubes before the units could be switched to synchronous mode. The tailwater depression system would be sized with enough storage tank capacity to complete the initial blowdown in a few seconds, and a smaller compressor could be used to maintain the draft tube pressure and water elevation over long periods of time.

Another consideration is the static pressure on the turbine head covers and main shaft seals due to the increased tailwater elevation. At the current normal tailwater elevation, the turbine head covers are only under pressure with the wicket gates open and the turbine units running. When the wicket gates are closed, the water in the turbine and draft tube equalizes with the tailwater and relieves the pressure on the head cover. If the tailwater elevation were higher than the turbine head covers, the head covers and associated seals would experience static internal pressures when the wicket gates were closed with the draft tube gates open. A tailwater depression system would maintain the same static pressure under the head covers while the units operated in synchronous mode. The unit centerlines are at elevation 1,073.0, which is 14.5 feet below the new maximum tailwater elevation of 1,087.5 feet. The 14.5 feet of static head is much less than the 204 feet of head that the turbines currently operate under. Therefore, the existing seals in the turbines should be capable of withstanding the static head at any of the proposed maximum tailwater elevations. Aside from the decrease in generation due to the reduced net head, the existing turbines should function properly with the new maximum tailwater elevation.

Reduction in Generation Capacity

Initial estimates of reduced generation at the Pit 7 Powerhouse used the maximum proposed pool elevations for Shasta Lake, but more recent models recognize that the maximum pool would not be maintained for extended periods, and may not even be achieved most years. The most recent estimates of reductions in energy generation at the Pit 7 Powerhouse indicate that annual generation would not be reduced by more than 2.6 percent assuming an 18.5-foot dam raise. The 12.5-foot dam raise would result in an estimated reduction in annual generation of 1.6 percent, and the 6.5-foot dam raise would result in a 0.8 percent reduction.

Analysis of the sump and dewatering pumps, cooling water systems, air intake systems, and wall penetrations can be found in the SLWRI *Pit 7 Dam and Powerhouse Facilities Report* (Reclamation 2008h).

Existing Powerhouse Structure

The existing Pit 7 Powerhouse has 4 foot-thick concrete walls. These walls span vertically between the concrete floor slabs to resist lateral soil and hydrostatic loads. A review of the structure's capacity indicates that the existing powerhouse wall would perform satisfactorily when subjected to loading from the maximum proposed tailwater elevation. The existing powerhouse structure would not require modifications to accommodate any of the proposed tailwater elevations.

The existing draft tube gates are fabricated from steel wide-flange beams and steel plate. A review of the gate elements indicates that under the new maximum tailwater elevation, the 11/16-inch-thick skin plate and the 24-inch-deep wide flange beams spanning horizontally that comprise the draft tube gates would perform adequately under the proposed tailwater elevations associated with the three Shasta Dam raise options. Therefore, the existing draft tube gates are considered adequate for continued service under the maximum proposed tailwater elevation. Further detailed review and/or analysis of specific elements or locations may be required at a later date, when a final tailwater elevation is determined.

Pit 7 Afterbay Dam

The Pit 7 Afterbay Dam consists of two main sections: a rockfill dam with a 550-foot-long crest with elevation 1,060 feet (NGVD29) (note that the following elevations presented in this section are based on the NGVD29 datum), and a concrete uncontrolled spillway section 145 feet in length, with a crest elevation that varies between elevation 1,026 feet and elevation 1,058 feet.

Slope stability analyses results for the Pit 7 Afterbay Dam, as modeled under the higher water levels cause by a raised Shasta Dam and Reservoir, show that the factors of safety against slope failures of the rockfill dam are above 2.0 for all the loading cases considered. Although the analyses show that complete failure of the rockfill dam would not occur under seismic loading, local surficial deformations would likely occur on the upstream side and less likely on the downstream reinforced face.

At the time of year when Shasta Lake is at the maximum normal pool, elevation 1,087.5, the rockfill dam is under 27.5 feet of water. The dam would be subjected to both hydrostatic and hydrodynamic forces that are not expected to have any significant impacts to the structural stability of the dam. The weight of water on top of the dam acts against slope failures and would increase the factor of safety. Flow velocities through the dam are not expected to be high enough to cause erosion of the dam. If high velocities were to occur, the steel reinforcement in the dam and on the downstream slope would act against erosion of the dam. Based on this feasibility level study, it is not recommended that any modifications be made to the Pit 7 Afterbay Dam. However, the steel reinforcement should be inspected regularly.

Keswick Dam

Keswick Dam is a concrete, gravity-type structure with a spillway over the center of the dam, and is located downstream from Shasta Dam. The spillway has four 50-foot by 50-foot fixed-wheel gates with a combined discharge capacity of 248,000 cfs at full pool elevation (587). It is estimated that no modifications to Keswick Dam would be required for the 6.5-, 12.5-, or 18.5-foot Shasta Dam raise options.

Probable Maximum Flood

An appraisal-level PMF was originally developed by Reclamation for the Shasta Dam Enlargement Studies in April 2001 (Reclamation 2001a, 2001b), using current Hydrometeorological Report No. 59 (HMR 59) procedures. The 2001 PMF was a general storm with a peak inflow of 633,400 cfs and a 15-day volume of 3,961,700 acre-feet, and was used to develop an appraisal-level hydrologic hazard analysis (HHA). The HHA provided frequency floods with return periods ranging from 100 to 20,000 years. Of note, the 20,000-year frequency flood was the same size (by peak and 15-day volume) as the appraisal-level PMF (Reclamation 2002).

A feasibility-level PMF hydrograph was developed by Reclamation for the dam raise feasibility design in January 2008 (Reclamation 2008d), with a peak inflow of 631,806 cfs and a 30-day volume of 6,245,905 acre-feet for a general storm on snow. The peak 15-day volume for the revised PMF was estimated to be 4,970,100 acre-feet. This corresponds closely with the appraisal-level PMF peak inflow, but is significantly greater in 15-day flood volume because of increased contributions from the Pit River Arm of the Shasta Lake drainage basin, and because of a generally larger (and longer duration) 100-year rain-on-snow antecedent flood. The Sacramento, McCloud, and lower Pit river subbasins would contribute the majority of the flow and would be responsible for the PMF peak inflow. The upper Pit River subbasins, with a total area of 3,310 square miles, were shown to contribute only about 2 percent to the peak inflow and 7 percent to the flood volume.

A revised HHA was prepared for the 2008 *Comprehensive Facility Review of Shasta Dam* by scaling the frequency flood hydrographs from the revised PMF (Reclamation 2008e). Table 3-12 summarizes peak discharge and volume estimates for return periods between 100 and 20,000 years. The revised 20,000-year frequency flood has about the same inflow peak and 15-day volume as the revised (feasibility-level) PMF.

Table 3-12. Revised Frequency Flood Peaks and Volumes, Shasta Dam

Return Period (years)	Peak Discharge (cfs)	Flood Volume (acre-feet)				
		1-Day	3-Day	5-Day	7-Day	15-Day
100	259,134	471,474	1,051,291	1,382,661	2,091,844	3,344,938
200	300,152	546,344	1,181,525	1,479,017	2,232,652	3,529,297
500	358,657	653,186	1,399,982	1,720,821	2,442,971	3,789,612
1,000	406,375	740,373	1,577,562	1,921,799	2,619,760	3,999,925
2,000	457,260	833,383	1,766,161	2,140,084	2,811,839	4,222,543
5,000	529,664	965,787	2,033,887	2,448,206	3,088,838	4,536,346
10,000	588,534	1,073,488	2,250,234	2,696,538	3,314,482	4,789,570
20,000	631,800	1,152,666	2,409,440	2,878,272	3,479,689	4,974,552

Key:
cfs = cubic feet per second

Flood routings were performed for both the existing and raised conditions, using an Excel spreadsheet to simulate the Emergency Spillway Release Diagram (ESRD) to properly limit discharges when the RWS is below the top of joint-use storage.

Borrow Areas/Sources

Construction at Shasta Dam for the main concrete dam, right wing dam, left wing dam, spillway, river outlets, power outlets, TCD, and visitor center will require nearly 100,000 cubic yards of concrete, or between 150,000 and 180,000 tons of sand and gravel, and between 20,000 and 25,000 tons of cementitious material (cement and pozzolan). Construction for the UPRR railroad bridges and Pit River Bridge modification will require nearly 23,000 cubic yards of concrete, or approximately 35,000 tons of sand and gravel, and approximately 6,000 tons of cementitious material. Construction for Lakeshore and Bridge Bay area embankments would need more than 300,000 cubic yards of core and homogenous fill, shell fill, riprap, filter, and drain materials.

Potential Borrow Sources

Multiple borrow sources are available to meet project needs for core and homogenous fill, shell fill, riprap, filter, and drain materials of the reservoir area embankments. Potential borrow sources were examined at a preliminary level and would need further sampling and testing to determine suitability and to refine quantity estimates. These borrow sources could include areas of the dike construction sites, areas located below the reservoir's inundation zone, and commercial sources. Material availability would vary with market demand and production restrictions, but it is expected that sufficient concrete sand and gravel materials will be available when needed for construction. More details regarding potential local sand and gravel aggregate sources are contained in Reclamation's *Shasta Dam Enlargement – Sand and Gravel Aggregate Sources* (2007c). For projects involving more than 100,000 cubic yards, Reclamation recommends locating five times the material required for construction when evaluating project feasibility to ensure adequate fill materials (Reclamation 1987).

Potential borrow sources and fill materials available at these borrow sources are summarized in Table 3-13. Commercial sources are located within approximately 2 to 30 miles of the Bridge Bay site, and within approximately 15 to 43 miles of the Lakeshore sites. Locations of potential reservoir area and commercial borrow sources are identified in Plate 25.

Table 3-13. Summary of Potential Borrow Resources

Borrow Sources	Core (cy)[1]	Shell (cy)[1]	Drain & Filter (cy)[1]	Riprap (cy)[1]
Shasta Dam Area	0	500,000	N/A	N/A
Lakeshore Drive Area	642,000	553,000	N/A	230,000
Bridge Bay Marina Area	N/A	674,000	N/A	200,000
Material Total	642,000	1,727,000	N/A	430,000

Note:
[1] Some of these volume estimates have not been field-verified by subsurface investigations.
Key:
cy = cubic yards
N/A = not available

The top 3 feet of material in all borrow areas are likely to contain organic matter, and be unsuitable for construction purposes. This surface material should be stockpiled and used to restore the sites after borrow activities are completed. A 3-foot increment was subtracted from the estimated depths of borrow materials to estimate volumes for all borrow areas.

Riprap material developed from moderately to slightly weathered basalt can be quarried below a thin soil cover in most locations along the Lakeshore Drive area. It is assumed that if a borrow area is selected for soil materials, this area could also be quarried for the required riprap.

Clean sand required for the proposed filters and drains was not found in any of the borrow locations investigated. Sand may possibly be produced from crushing and processing the basalt. Further study is recommended to evaluate the feasibility of crushing basalt into sand and gravel to construct drainage and filter elements.

For additional details on potential borrow sources, please refer to Reclamation's *Geology Report 2, Phase 1 Feasibility Geologic Report for Shasta Dam Enlargement Project* (2009b).

Borrow Recommendations

The borrow source evaluation was conducted as part of feasibility-level analyses for the Shasta Dam and Reservoir enlargement project for the purpose of feasibility design and planning. Note that the statements provided herein are based on available information and not on field explorations or laboratory testing.

Based on anticipated borrow requirements, and the initial assessment of potential borrow sources, construction of the proposed Lakeshore and Bridge Bay dikes is generally feasible with respect to the availability of construction materials. Note, however, that the findings in this section are based on feasibility-level analyses that may change as the project is developed. Design changes and field investigations may encounter conditions that would modify the evaluations and statements presented in this Engineering Summary Appendix.

It is recommended that geotechnical investigations be conducted to evaluate actual conditions present at the proposed borrow sites before final design of the proposed project. Laboratory testing would be required during subsequent phases of design to determine the actual physical and strength characteristics of borrow materials.

Geology/Geotechnical

The following sections discuss the regional geology and seismicity of the SLWRI study area.

Regional Geology

The regional geology of the area surrounding Shasta Dam is influenced by the intersection of three tectonic plates, defined as a triple junction (Reclamation 2007a). The Mendocino Triple Junction, composed of the Gorda, North American, and Pacific plates, is one of the most seismically active regions of the San Andreas transform system. Since 1983, the region has generated about 80 earthquakes with a magnitude of greater than or equal to 3.0 each year, and historically the region has experienced major quakes. This activity is generated in response to ongoing plate motions among the three plates of the Mendocino Triple Junction, which lies approximately 100 miles east of Shasta Dam (USGS 2007). Three active fault zones make up the tectonic interaction. These faults are the San Andreas Fault Zone, Cascadia Subduction Zone, and Mendocino Fault Zone. Both the San Andreas and Mendocino faults are classified as transform faults, or faults whose motion is normal along the fault trace. The Cascadia Subduction Zone is the result of the Pacific plate being subducted under the North American plate.

The interaction of the three plates of the Mendocino Triple Junction has created varied surface geomorphology, resulting in five major geomorphic provinces in the area of Northern California: the Klamath Mountain Range, Cascade Mountain Range, Great Valley Sequence, Modoc Plateau, and Coastal Mountain Range (Reclamation 2007a). Shasta Dam and Reservoir are located at the southeastern edge of the Klamath Mountains geomorphic province.

The Klamath Mountains geomorphic province is formed from the compressional and uplift forces of the volcanic arc and continental margin sequence (Reclamation 2007a). Surface topography is rugged, with prominent peaks and ridges reaching elevations 6,000 to 8,000. Throughout this mountainous region are thrust faults; structure relates to low-grade metamorphism. Rocks of the Klamath Mountains range in age from Ordovician to late Jurassic, and consist of greywacke sandstones, greenstones, cherts, limestone, and metamorphic equivalents of (the foregoing) rock types, and abundant granitic intrusive and ultramafic sheets.

Geology and subsurface foundation conditions for proposed structure modifications and new construction have been observed by Reclamation. A subsurface investigation performed in 2009 by Reclamation included fifteen boreholes 50 feet to 100 feet deep and fifteen test pits. These boreholes and test pits were located in the Shasta Dam, Lakeshore Drive, and Bridge Bay Marina areas to characterize subsurface foundation conditions within designated areas, document geologic investigations, and identify and characterize borrow sources (Reclamation 2009b). Known foundation conditions have been summarized on a dike-by-dike basis in Chapter 4.

Seismicity

The closest active fault to Shasta Dam is the Battle Creek Fault, located approximately 25 miles to the south. The Battle Creek Fault is a normal fault with a length of 29 kilometers (km), a dip of 75 degrees, and a width of 11 km (USGS 2002). The characteristic magnitude is 6.5, with a recurrence rate of 7.6 E-04/year and a slip rate of 0.5 millimeters (mm) per year.

Since 1973, 1,548 recorded earthquakes greater than magnitude 3.0 have occurred within 200 km of Shasta Dam. This radius includes earthquakes generated by activity in the Mendocino Triple Junction. When the search radius is reduced to 100 km (excluding the Triple Junction), only 224 earthquakes greater than magnitude 3.0 are found. The majority of earthquakes in this region are magnitude 3.0 to 4.0. The largest earthquake near Shasta Dam occurred on

November 26, 1998, and was magnitude 5.4 (USGS 2007). This earthquake was not located on the Battle Creek Fault.

Chapter 4 Design Consideration for Reservoir Area Infrastructure Modifications and/or Relocations

Road Relocations

As a result of the proposed Shasta Dam raise, existing reservoir area roads inundated by the increase in full pool elevation would need to be removed and/or relocated. The following sections discuss road relocation design criteria and estimated material quantities for each dam raise alternative.

Design Criteria Basis

Feasibility-level road replacement design criteria were established based on the following documents:

- *A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials* (AASHTO) (1990).
- *Shasta County Development Standards, Shasta County* (1997).

Research was conducted into the road design standards used by the U.S. Forest Service (USFS). Some information was provided by USFS from Chapter 4 of the USFS Road Preconstruction Handbook; however, no specific USFS requirements pertaining to road materials, widths, and construction were obtained. For this level of analysis, the criteria established below, based on AASHTO and Shasta County standards, are sufficient. Further research into USFS road requirements should be conducted before final design of roadways. For additional design and specification information, see the SLWRI Road Relocations Technical Memorandum (Reclamation 2007f).

Road Design Criteria

Criteria were established for four typical road replacement scenarios. (Table 4-1 presents the feasibility-level road design criteria. Graphical representation of the typical road replacement sections can be found in Plate 26.)

Nearly all paved roadways to be replaced have a paved width of approximately 18 feet, 20 feet, or 24 feet; therefore, typical paved sections of those dimensions were established. The design intent was to match road replacement widths with those of the existing paved roads to be replaced. The thicknesses used for asphaltic concrete (AC) and aggregate base (AB) are based on Shasta County typical requirements. During final design of the road relocations, geotechnical recommendations should be provided regarding actual AC and AB thicknesses to be used for construction.

Table 4-1. Typical Road Replacement Scenarios

Criteria	24-foot-wide Paved Road	20-foot-wide Paved Road	18-foot-wide Paved Road	Unpaved Road (AB)
Paved Width	24 feet	20 feet	18 feet	Varies
Shoulder Width	4 feet	4 feet	2 feet	2 feet
Roadside V-Ditch	As required	As required	As required	As required
Cross Slope	3%	3%	3%	3%
AC Thickness	4 inches	3 inches	3 inches	N/A
AB Thickness	8 inches	8 inches	8 inches	8 inches
Traffic Index	6.0	5.0	4.5	N/A
Rolling Terrain Minimum Design Speed	40 mph	30 mph	20 mph	20 mph
Mountainous Terrain Minimum Design Speed	30 mph	20 mph	20 mph	20 mph
Maximum Grade	8%	8%	10%	10%
Minimum Horizontal Curve Radius	50 feet	50 feet	50 feet	50 feet
Minimum Right-of-Way	60 feet	50 feet	50 feet	50 feet
Minimum Freeboard (above elevation 1,090.20 (NAVD88))	3 feet	3 feet	3 feet	3 feet

Key:
AB = aggregate base
AC = asphaltic concrete
mph = miles per hour
NAVD88 = North American Vertical Datum of 1988
N/A = not applicable

Unpaved roads requiring replacement vary in road width and in road material. Some unpaved roads have a gravel finished surface and others are earth with no finish material. For this analysis, it was assumed that all unpaved roads would be replaced with an 8-inch-thick AB section meeting the width of the existing road.

Guardrail would be placed in areas where the existing road has guardrail or in areas where conditions have a higher potential for vehicles to run off the road.

Culverts would be placed at low points, as required, based on the road relocation grading designs. In areas where feasible, low points would be filled with embankment fill rather than providing a culvert. The majority of existing culverts in the Shasta Lake area appear to be corrugated metal pipe (CMP); thus, this was the pipe material assumed for the analysis. Minimal analysis was conducted to establish the culvert diameters shown. Acreages of the upstream stormwater sheds were estimated, and a culvert diameter was assumed based on these areas. These culvert sizes should be used for feasibility-level cost estimating only. During final design of the road relocations, a complete hydrological analysis should be conducted to size all culverts for the established design storm.

Properly graded riprap is commonly used to provide slope protection. Riprap would be placed on new embankment slopes that would be subject to wave erosion. Riprap would need to be placed in a manner that would provide a well-integrated mass with minimum void spaces. Generally, riprap consists of a uniform distribution of angular and durable gravel- to boulder-sized rock.

In areas where it would not be feasible to make steep hillside excavations or use embankment fill to establish the finished road grades, cast-in-place (CIP) concrete retaining walls, with spread footings, would be used. Wall dimensions are based on previous retaining wall designs. This detail should be used for feasibility-level cost estimating only. During final design, the need for retaining walls should be revisited and designed accordingly for construction.

The existing terrain around Shasta Lake varies from rolling to mountainous, depending on the specific area. Slopes of the existing hillside embankments typically range from as steep as 0.25:1 to as flat as 3:1. Slopes of the existing terrain were assessed at each road relocation area during the analysis. For this analysis, it was assumed that the steepest cut slope allowed would be 1:1 and the ideal cut slope would be 2:1. If an existing roadway cut slope appeared to be in good condition, it was assumed that the new road cut slopes in the same vicinity could safely match the existing conditions. The steepest fill slope permitted would be 1.5:1 and the ideal fill slope would be 2:1. Borrow material for road embankment construction would be derived from sources identified in Chapter 3.

Estimated Quantities

Feasibility-level quantities, based on the feasibility-level designs, have been generated for each road segment. It is estimated that approximately 4.1 miles of paved roadway and 2.3 miles of unpaved roadway would need to be constructed as a result of the new full pool water surface elevation. Table 4-2 summarizes the estimated quantities. Imported fill borrow areas would be similar to those outlined for dikes and embankments in Chapter 3.

Table 4-2. Summary of Approximate Road Relocation Quantities

Dam Raise (feet)	Length (lineal feet)	Paved Area (square feet)	Embankment Fill (cubic yards)	Excavation (cubic yards)
6.5	17,409	322,854	259,400	65,115
12.5	29,054	542,614	396,521	78,270
18.5	33,788	630,314	424,121	82,070

Relocations for 6.5-foot and 12.5-foot Dam Raises

These analyses were performed to support the 18.5-foot dam raise feasibility. However, 6.5- and 12.5-foot dam raise scenarios have been considered in the past. As a result of a 6.5- and 12.5-foot Shasta Dam raise, the new reservoir full pool elevations would be 1,078.2 (NAVD88) and 1,084.2 (NAVD88), respectively. Table 4-3 summarizes the road segments that would be inundated by the 18.5-foot dam raise, and continue to be inundated by 6.5- and 12.5-foot dam raises. It is estimated that approximately 1.5 miles of paved roadway and 0.8 miles of unpaved roadway would have to be constructed as a result of a 6.5-foot dam raise; 2.5 miles of paved roadway and 1.5 miles of unpaved roadway would have to be constructed as a result of a 12.5-foot dam raise.

Conclusions

The major road and road segment feasibility-level designs presented in this Engineering Summary Appendix are potential solutions to the inundation of existing roads as a result of an 18.5-foot dam raise. As work continues toward a more detailed design phase of these road modifications, it is recommended that the following be considered and data be obtained:

- Further coordination must be conducted with USFS and Shasta County. Consideration should be given to the need for some of the road segments to be above the new full pool elevation.

- Further coordination must be conducted with UPRR regarding the Lakeshore Drive relocation impacts on the existing and relocated UPRR tracks in the Lakeshore area.

- A current topographical survey for each road segment area would need to be conducted before final design.

- Geotechnical data and recommendations would need to be obtained specific to each road segment area, including, but not limited to clearing and site preparation, slope stability, excavation and shoring, subgrade preparation, embankment fill materials, compaction criteria, retaining wall earth pressures, and pavement sections

- Additional design refinements and cost reduction ideas would need to be evaluated and integrated, as appropriate.

Table 4-3. Impacts from 6.5-Foot and 12.5-Foot Dam Raises

Road Segment ID No.	Description	6.5-Foot Dam Raise Impact (Y or N)	12.5-Foot Dam Raise Impact (Y or N)
32	Unpaved road to waterfront	Y	Y
71	USFS Road 35N17–Conflict Point	N	N
76A	USFS Road 35N08–Lakeshore Drive	Y	Y
76B1	USFS Road 35N08–Lakeshore Drive	N	N
76B2	USFS Road 35N0–Lakeshore Drive	Y	Y
76C	USFS Road 35N08–Lakeshore Drive	N	N
77	USFS Road 35N17–Conflict Point	N	Y
81	USFS Road 35N08–Lakeshore Drive	Y	Y
93	Shasta County Road–Lakeshore Drive	Y	Y
99	Shasta County Road–Lakeshore Drive	N	Y
110	Shasta County Road–Lakeshore Drive	N	Y
109	USFS Road 35N14E–Antlers Road	N	N
34	USFS Road 34N09B–Lower Deck	Y	Y
39	USFS Road 34N09A–Shasta Yacht Club Road	Y	Y
96	USFS Road 35N60D–Hirz Road	N	Y
135	Shasta County Road 7H009–Gillman Road	N	Y
140	Shasta County Road 7H009–Gillman Road (McCloud River Bridge)	N	Y
141	Shasta County Road 7H009–Gillman Road (McCloud River Bridge)	N	Y
144	USFS Road 36N54–Bollibokka Club Road	N	N
174	USFS Road 34N09–Turntable Road	Y	Y
7	USFS Road 33N13–Jones Bay	Y	Y
9	USFS Road 33N86–Jones Valley	N	N
20	Shasta County Road 5J050–Silverthorn Road	N	N
21	Shasta County Road 5J050–Silverthorn Road	N	N
156	South Access Road–Pit River Bridge	Y	Y
41	USFS Road 35N03–Salt Creek Road	N	N
47A	USFS Road 35N03–Salt Creek Road	N	Y
47B	USFS Road 35N03–Salt Creek Road	N	Y
61	USFS Road 35N03–Salt Creek Road (Didallas Creek Bridge)	N	Y
62	USFS Road 35N03–Salt Creek Road (Didallas Creek Bridge)	N	Y

Key:
ID = identification
N = no
USFS = U.S. Forest Service
Y = yes

Utilities and Miscellaneous Minor Infrastructure

As a result of the proposed Shasta Dam raise, existing infrastructure inundated by the increase in full pool elevation would need to be removed and/or relocated. The following sections discuss existing reservoir area utilities, design criteria, and relocations approach.

Existing Utilities and Minor Infrastructure Description

As previously stated, infrastructure was inventoried around the perimeter of the reservoir that would be impacted by the proposed 18.5-foot dam raise (Reclamation 2007h). This 18.5-foot dam raise corresponds to a 20.5-foot raise in the full pool because of associated modifications in operation of the dam (new full pool elevation 1,090.2, NAVD88). The identified inventory items included bridges, buildings, dams, gas/petroleum facilities, hazardous materials, miscellaneous objects, parking areas, power towers, railroads, and roads. Also on the inventory are potable water, power distribution, telecommunication, and wastewater facilities.

A majority of the infrastructure adjacent to Shasta Reservoir is located along the Interstate 5 corridor (see Plates 1 and 2). The largest potentially impacted residential developments near the reservoir are in the Lakeshore and Sugarloaf areas in the northern part of the Sacramento River arm (see Plate 2). The main facilities in the Pit River arm are at Bridge Bay Marina and in the Jones Valley and Silverthorn areas. The upper Pit River arm is very remote; the only significant infrastructure is the Fender Ferry Bridge and Pit 7 Dam at the upstream end. Main development along the McCloud River arm includes several USFS campgrounds and a few marinas, the Bollibokka Club, and some summer-use cabins. The Squaw Creek Arm has the least infrastructure, with the old Bully Hill Mine and a few cabins.

Only impacts to gas/petroleum, potable water, power distribution, telecommunications, and wastewater facilities will be discussed in this Engineering Summary Appendix; other types of facilities may be listed in the tables for completeness but are not discussed in detail. Following is a description of existing facilities and a discussion of their distribution around the perimeter of the lake.

Gas/Petroleum Facilities

The inventory concluded that no natural gas facilities are present in the inventory area (Reclamation 2007h). Home heating and cooking gas used in the area are exclusively propane. Propane tanks for homes and businesses were not included in the inventory because they are portable and also may be leased. The gas/petroleum facilities that were identified were primarily gasoline and diesel fuel storage tanks. The majority of the tanks were used to store fuel for boats. The tanks varied in size from approximately 1,000 gallons to 4,000 gallons.

Potable Water Facilities

Potable water is provided in one of three ways in the inventory area: (1) water may be provided by Shasta County through county service areas (CSA); (2) by mutual water companies; (3) by individual residence or group wells. The two CSAs in the inventory area are CSA 2, which operates in the Sugarloaf community, and CSA 6, which provides water to the Silverthorn summer homes. Mutual water companies are cooperative or mutual associations that furnish water to a resort or development. Fifteen mutual water companies were identified in the inventory area. Wells serving groups of homes or resorts were also identified in the inventory (Reclamation 2007g). Individual homes or businesses that were not confirmed to be associated with CSAs, mutual water companies, or group wells were assumed to have an individual well.

Power Distribution Facilities

All electric power service in the inventory area is provided by PG&E. Power lines are typically routed overhead on poles or towers, although a portion of the lines serving individual businesses, homes, and cabins is routed underground. Power lines are also frequently attached to bridges when routed over rivers and lake inlets. Voltage of local distribution lines is typically 12 kilovolts (kV) while the voltage of high-voltage transmission lines is typically 60 kV to 230 kV. Service to individual homes and businesses is typically 120 to 480V.

Telecommunications Facilities

Telecommunication services in the inventory area are primarily provided by American Telephone and Telegraph (AT&T). Qwest Communications was listed with AT&T on one line but the continued ownership was not confirmed. One cable television operator, DCA Cable, was reported to have facilities in the inventory area but this was not confirmed and these facilities were not located. Telecommunication lines in the area are either copper wire or fiber optic cable, which, similar to power lines, may be overhead, underground, or attached to bridges. It should be noted that no cell phone towers were identified in the inventory area (Reclamation 2007g). Also, AT&T confirmed that there were no transcontinental fiber-optic lines in the inventory area.

Wastewater Facilities

No large wastewater collection or treatment systems are located in the inventory area. Wastewater treatment is accomplished using septic tank/leach field systems or vault/pit toilets. At several larger resorts, three to five cabins or buildings were routed to a single septic system. In all other cases, individual homes, cabins, or businesses were routed to individual septic systems. Campgrounds and public restrooms were either septic tank/leach field systems or vault/pit toilets (Reclamation 2007g).

Design Criteria

The following sections discuss the demolition and design criteria for relocated utilities and minor infrastructure within the SLWRI study area.

Demolition Criteria for Existing Utilities and Minor Infrastructure

A set of criteria was established for each category of utilities to determine whether or not the utilities would need to be demolished.

Buildings Knowledge of the relocation status of all potentially impacted buildings is critical to accurately assessing the extents of utilities and minor infrastructure that would require removal or relocation. In addition, an accurate accounting of the extent of relocated utilities and minor infrastructure cannot be determined unless the ultimate location of the relocated buildings is known.

If buildings are inundated, it will be assumed that all utilities associated with that building would be completely demolished or, in the case of linear utilities, demolished to the point where the utilities are also associated with another building that is not being demolished or relocated.

Residences Any residence that is inundated or within 3 vertical feet of the inundation line would be demolished along with its associated utilities unless otherwise specified. Demolished residences would not be relocated.

Recreation Any recreation building that is inundated or within 3 vertical feet of the inundation line would be demolished along with its associated utilities. All demolished recreation buildings and their associated facilities would be relocated, with the exception of recreation facilities that would be abandoned.

Septic Systems
Demolition Criteria The Shasta County Development Standards state the following (Shasta County 1997):

> *Disposal area shall not include:*
>
> *Land closer than 200 feet to a lake or reservoir, measured from the high water line or 100 feet if down slope from the lake or reservoir.*

These criteria would be applied to the septic system demolition criteria, indicating that septic systems within 200 feet of the new full pool waterline or 100 feet downslope of the new full pool waterline would be demolished.

Demolition Practices

Wastewater Pipes Abandon in place wastewater pipes 6 inches in diameter and smaller. Fill pipes larger than 6 inches with sand and abandon in place per Shasta County Public Works Department requirements (Shasta County 1997).

Septic System and Vaults/Pits Pump out septic tank, fill with sand, and abandon in place per Shasta County Environmental Health Division requirements (Shasta County 1997). Pump out vaults/pits, fill with sand, and abandon in place. Abandon leach fields in place. Demolish associated restroom building and contents and take to an approved landfill. A Shasta County permit is required.

Water System

Demolition Criteria The water systems affected by the inundation line consist of either wells or waterlines.

Wells Several wells do not fall within the inundation line but are close to the shoreline at the new full pool. If these wells appear to be associated with buildings that would be demolished, they would also be demolished. If these wells are associated with a water system or with buildings that would not be demolished, they would remain in place.

Waterlines Waterlines that would be relocated would be moved to a minimum of 20 feet from the shoreline at the new full pool. Several waterlines do not fall within the inundation line but are close to the shoreline at the new full pool and are located at a depth below the water level. These water lines would remain in place.

Demolition Practices

Water Pipes Abandon in place water pipes 6 inches in diameter and smaller. Fill pipes larger than 6 inches with sand and abandon in place per Shasta County Public Works Department requirements (Shasta County 1997).

Wells Fill wells with sand and abandon in place per Shasta County Environmental Health Division requirements (Shasta County 1997). A Shasta County permit is required.

Pump/Lift Station Demolish building and contents and take to an approved landfill. Abandon associated underground piping in place. Reseed area.

Power and Telecommunication Facilities

Demolition Criteria Most power lines and telecommunication lines are along the same alignment because they typically use the same power pole. The majority of the power lines and telecommunication lines are overhead with a few underground lines.

Any low-voltage power lines, telecommunication lines, or power poles that are inundated or within 50 feet from the new full pool elevation would be relocated a minimum of 50 feet from the new full pool elevation.

Any high-voltage power lines or power towers that are inundated or within 100 feet from the new full pool elevation would be relocated a minimum of 100 feet from the new full pool elevation.

Demolition Practices

Power and Telecommunication Lines Demolish power and telecommunication lines in accordance with the National Electrical Safety Code (NESC) and California Public Utilities Commission General Order 95 (CPUC-GO 95) requirements. Remove all poles and wires and dispose of at an approved landfill. Excavate and remove connection point to underground wires to 30 inches below grade and abandon remaining underground wires in place.

Gas/Petroleum Facilities and Miscellaneous Objects

Demolition Criteria Demolish and relocate any gas/petroleum facilities and/or miscellaneous objects that are inundated.

Demolition Practices

Fuel Tanks Excavate and remove existing underground tanks and all associated piping. Perform hazardous material testing and removal, as required, in accordance with Title 23 of the California Code of Regulations, Division 3, Chapter 16, Underground Tank Regulations (State of California 2005), and in accordance with Shasta County Environmental Health Division requirements (Shasta County 1997). A Shasta County permit is required.

Design Criteria and Assumptions for Relocated Utilities and Minor Infrastructure

Facilities to be relocated would be designed and constructed in accordance with all applicable Federal, State, and local codes and requirements. Demolished facilities would not be reused to construct relocated facilities. Relocated facilities would be of the same types, sizes, and materials as the existing facilities to be replaced, where in compliance with applicable codes and requirements. Additional criteria for specific facilities are discussed in the following sections.

Gas/Petroleum Facilities Relocated fuel storage tanks would be designed and constructed in accordance with Title 23 of the California Code of Regulations, Division 3, Chapter 16, Underground Tank Regulations (State of California 2005); the Uniform Fire Code (NFPA 2006); California Air Resources Board, Shasta County Development Standards, Section 6.7, and Shasta County Environmental Health Division requirements (Shasta County 1997).

Potable Water Facilities Relocated potable water facilities would be designed and constructed in accordance with Shasta County Development Standards, Chapter 7, and the following:

- Relocated wells would acquire and would meet conditions of a Shasta County Environmental Health Division Well permit.
- For cost estimating purposes, wells would be assumed to be 200 feet deep and produce 15 gallons per minute (gpm).

- Water main piping smaller than 4 inches in diameter would be American Society for Testing and Materials (ASTM) 1785 Schedule 40 PVC. Water main piping 4 inches in diameter and larger would be PVC – American Water Works Association (AWWA) C900, Class 150, standard dimension ratio (SDR) 18, or Ductile Iron – AWWA C151, Class 51 or 50.

- Water main piping serving fire hydrants would be a minimum of 6 inches in diameter, or as required.

- Water main piping serving only service connections would be a minimum of 2 inches in diameter, or as required.

- Individual service connections are assumed to be 0.75-inch- or 1-inch-diameter pipe.

- Blowoffs would be provided at all low points and on any main with dead-ends more than 10 feet past a fire hydrant.

- Combination air valves would be installed on all high points. Valves would be a minimum of 1 inch in diameter or as required.

- Minimum depth of cover would be 3 feet for water mains.

Power Distribution Facilities All safety and operational requirements for relocated power lines would comply with NESC and CPUC-GO 95 regulations. Underground lines would have a minimum of 30 inches of cover.

Telecommunications Facilities All safety and operational requirements for relocated telecommunication lines would comply with NESC and CPUC-GO 95 regulations. Underground lines would have a minimum of 30 inches of cover.

Wastewater Facilities Relocated wastewater facilities would be designed and constructed in accordance with Shasta County Development Standards, Chapter 5 (Shasta County 1997), and the following:

- Relocated septic systems would acquire and meet conditions of a Shasta County Environmental Health Division Sewage Disposal System Permit.

- For cost estimating purposes, septic tanks would be assumed to have an 1,100-gallon capacity, and leach fields would be assumed to be 100 feet long and 3 feet deep by 3 feet wide. Septic tanks would have a minimum of two compartments.

- Pipe would be PVC ASTM 3034, SDR 35 with Ring-Tite or Fluid-Tite joints, and would be a minimum of 6 inches in diameter, or as required.

General Facility Relocations Approach

The following sections discuss the general facility demolition and design criteria for relocated utilities and minor infrastructure within the SLWRI study area.

Fuel Storage Tanks

Relocated fuel tanks would be designed and constructed in accordance with Title 23 of the California Code of Regulations, Division 3, Chapter 16, Underground Tank Regulations (State of California 2005); Uniform Fire Code (NFPA 2006); California Air Resources Board; Shasta County Development Standards, Section 6.7; and Shasta County Environmental Health Division requirements, as noted in Section C (Shasta County 1997). The tanks would be located in cleared areas with code-mandated clearances from other facilities. For reference, 4,000-gallon capacity tanks may be assumed to be approximately 8 feet in diameter and 15 feet long.

Potable Water and Wastewater Piping

Relocated piping for potable water and wastewater facilities would follow the design criteria previously described and would typically be located within established roadways consistent with Shasta County Development Standards (Shasta County 1997). Pipe trench sections would meet Shasta County Development Standards (Shasta County 1997). Typical construction characteristics for potable water and wastewater piping are also described in the *SLWRI Utilities and Miscellaneous Minor Infrastructure Technical Memorandum* (Reclamation 2007g).

Potable Water Wells

Potable water wells would be designed and constructed per Shasta County Environmental Health Division requirements (Shasta County 1997). Wells would be sited with the required separation from septic systems, would be drilled to an approximate depth of 200 feet, and would have a capacity of approximately 15 gpm. The actual depth and capacities of specific wells would depend on groundwater table elevation and soil permeability characteristics at the particular site. Residential wells would typically include a bladder tank and a small well pump.

Wastewater Treatment Facilities

An expanded discussion of the approach and methodology for relocation of wastewater treatment facilities is presented below:

General Considerations With the proposed full pool water surface raised approximately 20.5 feet to elevation 1,090.2 (NAVD88), a number of wastewater facilities, primarily septic systems, would be directly inundated. In addition, some existing septic systems would no longer meet Shasta County Environmental Health Division requirements for separation from the lake (Shasta County 1997). This is significant because homes and businesses without permitted septic systems cannot be occupied and would require abandonment and demolition.

The approach to relocation of wastewater treatment facilities in the project area is to either construct new septic systems on the property of an impacted home, where feasible, or to define a possible wastewater treatment plant alternative to abandoning homes that do not meet Shasta County requirements for septic system separation from the lake. New septic systems would be constructed per Shasta County Development Standards (Shasta County 1997). Tanks would have two compartments and would be approximately 11 feet long, 5 feet high, and 5 feet wide, and buried with about 3 feet of cover. Leach fields would typically be 100 feet long and 3 feet wide by 3 feet deep. The actual size of specific septic tanks and leach field systems would depend on the size of the homes or businesses and the percolation characteristics of soils in the leach field area.

The locations where the possible wastewater treatment plant approach would be implemented are described in Section E of the *SLWRI Utilities and Miscellaneous Minor Infrastructure Technical Memorandum* (Reclamation 2007g). The general wastewater treatment plant concept includes a pressurized sewer collection system to transport wastewater flows to a number of centralized package wastewater treatment plants.

Plant Siting Considerations The following discussion defines wastewater treatment plant siting considerations, one package wastewater treatment plant alternative, other possible alternatives, and various assumptions that were made. Treatment plant siting is generally based on the following factors:

- Distance from the effluent discharge location. Locations as close to the effluent discharge location as possible are desirable.
- Treatment plant equipment layout (size of lot needed).
- Method of wastewater collection (gravity versus pumped). Maximizing gravity flows to minimize the pumping requirements is desirable.
- Environmental impacts to the potential site. Environmental impacts to the receiving water body, especially how the water quality management plan of the area is affected.
- Availability of power and other utilities.
- Accessibility to roads.
- Flooding potential and earthquake fault locations.
- Construction costs of building at a particular site.
- Site maintenance, including parking.

- Operator safety and neighborhood safety.

Specific plant siting issues applicable to the proposed project are described in this paragraph. It is generally assumed that most of the wastewater collection system (including individual laterals to homes) would need to be pumped because of the mountainous terrain and the likelihood that few sites suitable for pump stations or wastewater treatment facilities are located near Shasta Reservoir. If suitable sites are available near Shasta Reservoir, this would decrease pumping costs. No attempt has been made to evaluate the water quality impacts of wastewater treatment effluent discharge to Shasta Reservoir.

Odor control is another important factor to consider for plant siting. Odors from the wastewater facilities have the potential to affect nearby residents. Buffer zones are often used to deal with this problem. A buffer zone is a defined distance from wastewater facilities to the closest neighbor. This distance is determined by performing odor studies that take into account an odor source and strength, meteorological/dispersion conditions, and type of surrounding development. Buffer zones can be as large as 1,500 feet. Physical/mechanical methods may also be used to control odors. These methods would typically include carbon air filters combined with air blowers. If no physical/mechanical odor control is included in wastewater treatment design, this will increase the land area required. The approximate amount of land area required for each of the assumed treatment plants is about 0.25 acres. If a 1,500-foot-wide buffer zone were required, this area would increase to about 40 acres.

Wastewater Treatment Process The wastewater treatment process assumed in the current analysis is shown in Figure 4-1.

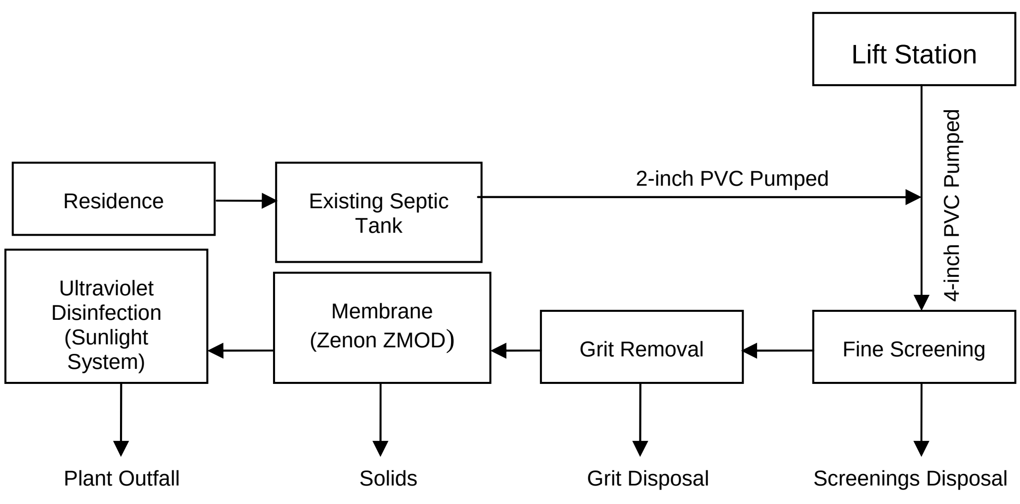

Figure 4-1. Wastewater Treatment Process Flow Schematic

The wastewater treatment process shown in Figure 4-1 is described in this paragraph. It is assumed in the proposed plan that the existing septic tanks at each residence can be reused. The tank discharge pipe to the disposal field would be capped off and the disposal field would become inactive. Sewage ejector pumps would be installed in each septic tank and the contents of the septic tank would be pumped via a 2-inch PVC line into a 4-inch PVC force main (pressurized sewer line). The 4-inch PVC force main for each system is assumed to be a common collection force main that would be installed in the existing roadways. It is assumed that lift stations would be required to pump all wastewater because of the mountainous terrain. Each lift station would consist of a 4-foot-diameter manhole with two grinder pumps. The footprint area required for each lift station would be approximately 15 feet by 15 feet. Two pumps would be required to provide redundancy in case one of the pumped stops operating. The 4-inch PVC force main contents would be pumped to the wastewater treatment plant. The wastewater treatment plant would require a fine screen (2 mm openings) to screen out all large solids to prevent damage to the proposed membrane system. Grit removal would follow the fine screens to remove small abrasive materials (e.g., sand) that can also damage the membrane system. Flow after these two preliminary treatment processes would then be directed to the secondary treatment process, which is an ultrafiltration membrane. This is basically a biological filter that removes smaller particles and organics. The last step in the treatment process would be ultraviolet (UV) disinfection, which is used to destroy disease-causing organisms before discharge. Treated water from this treatment train would then be discharged to Shasta Reservoir (pending permit approval). It has been assumed that this would be accomplished via gravity, although the mountainous terrain may require effluent to be pumped to Shasta Reservoir.

It is assumed that the system described above would be used at several locations around the lake.

Various issues should be understood when considering a small package treatment plant, as described above. Wastewater treatment in small communities is relatively expensive because the same level of treatment is usually required for small communities, without the financial advantage of the economy of scale of a larger community. Smaller communities are generally more spread out, which also increases the wastewater collection system cost. In the case of Shasta Reservoir, the mountainous terrain would increase the per capita cost even more. In addition, operation and maintenance costs that currently are not incurred would be required for the wastewater treatment system shown in Figure 4-1**,** further increasing the overall cost. Another issue is odor control. As discussed previously, buffer zones are often used to mitigate odor problems, and this has been assumed in the wastewater treatment process described above. If physical/mechanical odor control facilities would be required, this would increase a wastewater treatment facility’s capital cost and operation and maintenance costs.

Other Wastewater Treatment Alternatives It should be noted that other alternatives could be considered during final design, as described below.

Centralized Septic System This type of system would use existing septic tanks for solids separation, and then all septic tanks would pump septic tank effluent to a centralized septic field. The size of the septic field would be based on many different factors, including soil type and trench capacity. Trench capacity can include sidewalls or just the trench bottom. A rough calculation with no site-specific data, and the assumption of a wastewater flow capacity of 25,000 gallons per day (gpd), yields a septic field size of about 30 acres. If sidewalls are included in the calculation, the required area may be reduced to about 25 percent of the trench-bottom-only area.

Recirculating granular-medium filter In these systems, septic tank effluent is pumped to a sand filter multiple times before being discharged to a plant outfall or septic field. Disinfection is required if discharging to surface waters directly from the sand filter.

Other package treatment plant alternatives Many other package wastewater treatment plant alternatives could be suitable.

Power Distribution Facilities

An expanded discussion of the approach and methodology for relocating power distribution facilities is provided below.

General Considerations With a dam raise of 18.5 feet, several transmission and distribution lines would either be inundated or in close proximity to the reservoir. All transmission towers, power poles, and underground power lines that would be inundated would be relocated. There are no regulations pertaining to how far power lines need to be located from the inundation line. Underground power lines, transmission towers, and power poles merely need to be located in dry areas. It is assumed that areas 100 feet inland from the inundation line would be dry enough for the transmission towers, while areas 50 feet inland from the inundation line would be suitable for underground power lines and power poles. If a transmission line is within 100 feet of the inundation line, it would be relocated. If a distribution line is generally within 50 feet of the inundation line, it would be relocated. If a home is within 50 feet of the inundation line, and is not demolished because of other criteria, the local distribution line to the house would be retained.

Inventory of Existing Transmission and Distribution Lines in Project Area
An initial inventory of and data collection for the existing transmission and distribution lines near the reservoir, or subject to be inundated, was carried out to identify the volume of work that would be generated by the project.

Two 230 kV power towers (186, 187) and two 115 kV (3497, 3498), respectively, would be inundated. Approximately 3,000 feet of 230 kV and 5,900 feet of 115 kV transmission line would be demolished and relocated.

For the main distribution lines, about 23,300 feet would be demolished and relocated. An average distance of 75 feet was assumed for the distance between a home and main distribution line. Approximately 8,000 feet of local distribution lines would be demolished and relocated.

Selection of New Power Line Routes Route selection is a critical step in the design process for a transmission line. Any route selected would have some impact on the land and its uses. Therefore, it is important to analyze all possible alternatives and choose a route that would minimize those impacts, and allow construction, and facilitate utility personnel in easily building or accessing the routes.

From an engineering perspective, several factors associated with relocating power lines are typically addressed in a route selection study. These factors are economics, reliability, and environmental impacts. The power towers preferably would be located in dry areas where foundations are above the water table level and soils are capable of sustaining the foundations. If wetlands or severe terrains cannot be avoided, costly foundations and/or special construction equipment or techniques would be required. Also, crossing heavily forested areas that would require extensive clearing would be avoided, as well as hunting grounds with a high probability of gunshot damage to insulators. These are a few aspects to be considered when planning a new route.

Clearances Clearances for transmission lines are those required to provide safety to the public and those required to provide reliable operations. Clearances required for safety and, to some extent, for operations, are stated in the NESC. These include clearances above the ground, streets, railroads, etc. For transmission lines of 230 kV and below, NESC clearances would generally provide for reliable operations as well as safety. These clearances are based on the voltage between the phase conductor and ground (L-G voltage). When considering phase voltage, a multiplier of 1.05 should be applied to the nominal voltage to determine maximum L-G voltage. Structure clearances are concerned with the air gap clearance between an energized conductor and the supports to prevent lightning and switching surge flashovers. Table 4-4 shows clearances between a restrained conductor and supporting structure. If conductor support is not restrained, clearance would be adjusted according to the maximum design range of swing of the insulator string. Any additional clearance to allow workers movement during live-line maintenance work should be added to determine the minimum tower-to-conductor clearance.

Table 4-4. Restrained Conductor Clearances

Voltage (L – L)	Phase to Ground Voltage	Minimum Structure Clearance (feet)
69 kV and below	41 kV	1.3
138 kV and below	84 kV	2.5
230 kV and below	140 kV	4

Key:
kV = kilovolt
L – L = line to line

Clearances not covered by the NESC that should be considered are phase-phase clearance, for very long spans, galloping, and leakage current limitation.

At the start of the design of a line, it is important to develop a table of clearances as part of the design criteria to be used throughout the design. The table would include not only phase-to-ground and phase-to-phase structure clearances, but also phase-to-ground or crossing, or to the edge of right-of-way (ROW) clearances.

Also, various clearance requirements would need to be implemented from CPUC-GO 95. Since PG&E owns the power lines and ROW, PG&E criteria would also need to be met.

Rights-of-Way When considering a new line, it is always necessary to determine ROW width. It is known that transmission lines generate electric and magnetic fields, but no clear evidence has been found of any significant environmental and health effects resulting from the operation of these lines. However, some states have established regulatory limits on the strength of electric and/or magnetic fields from power lines. Therefore, a ROW easement obtained for transmission lines generally prohibits the installation of any buildings or facilities within the ROW, but would allow buildings or structures at the edge of a ROW. Therefore, the first condition for ROW width is to provide NESC horizontal clearances to buildings for the longest span in the transmission line, remembering that the clearance must be maintained with a 6-pound wind on the conductor and structure. Therefore, conductor swing and structure deflection must be considered when calculating the clearance. The minimum clearance under these conditions can be based on the basic impulse level (BIL) flashover distance rather than the NESC safety distance.

Table 4-5 presents typical minimum ROW widths for transmission lines with voltages (phase to phase) of up to 230 kV. However, ROW widths would need to be calculated based on site-specific data for any particular line.

Table 4-5. Minimum Right-of-Way Widths

Voltage (L – L)	BIL Flashover	Minimum Right-of-Way Width (feet)
68 kV and below	350 kV	40
138 kV and below	550 kV	60
230 kV and below	750 kV	75

Key:
BIL = basic impulse level
kV = kilovolt
L – L = line to line

Plan and Profile, Structure Spotting Plan and profile information is essential to determine structure locations. These drawings generally consist of a plan on half of a drawing and an elevation of a transmission centerline on the other half of the drawing. Such information is obtained by performing a survey along a transmission line corridor. This survey may consist of a ground survey or an aerial survey. Generally, short lines (such as the lines to be relocated by this project) use a ground survey, while long lines use aerial surveys.

When plan and profile information is prepared, it would include plan information and centerline elevations, as well as the following information:

- Section line locations
- Property line locations
- Property owners
- Road and street ROW
- Railroad ROW
- Wire crossings (e.g., transmission lines, telephone)
- Elevation of wire crossing
- Underground facilities (e.g., pipelines, cables)

Poles and Structures Poles can be of wood or steel, and used as a single support, or as a framed structure, such as an H-frame steel pole, unlike wood poles can be fabricated to almost any required strength and are therefore more versatile for longer span applications.

Regarding structures, even though some can be of wood, steel is the most common material, particularly for structures subjected to high mechanical loads. Lattice steel structures have been used since the earliest transmission lines, but more recently tubular steel poles have become common.

Foundations Wood pole or steel pole foundations can be directly embedded in the ground with crushed rock or concrete backfill, or installed using reinforced-concrete caissons and anchor bolts. The direct-embedment type foundation is acceptable for tangent and small-angle structures, but concrete caisson foundations would be required for large angle and dead-end structures.

Foundations for lattice steel structures are typically concrete caissons or earth grillages.

Permits A series of permits would need to be obtained from Federal and State agencies, and also from local governments for relocating power lines. As a minimum, the following agencies would be contacted and/or permits would be required:

State

- California Public Utility Commission (CPUC)
- California Department of Fish and Game (DFG)
- Caltrans
- Road crossing permit
- Railroad crossing permit

Municipal

- Neighborhood plan review from each town or community where a power line would cross a roadway
- Local permits for construction
- Local zoning permits or conditional use permits

Please see Environmental Impact Statement, Chapter 26, for additional information on compliance with applicable laws, policies, and plans.

Telecommunication Facilities

Relocation of telecommunication lines would comply with all NESC and CPUC-GO 95 requirements. Telecommunications lines would typically be attached to power poles at a height lower than the power lines, typically 18 feet off the ground. Buried lines would typically be routed along roadway shoulders and direct-buried with a minimum of 30 inches of cover.

Bridge Relocations

Raising Shasta Dam would affect nine bridges around the reservoir to varying degrees. Some bridges would require complete removal and replacement, while others would require protection of the piers from inundation. The bridges affected by a dam raise were identified as vehicle bridges or railroad bridges. Feasibility designs and costs were developed based on initial analyses performed in previous SLWRI milestone reports.

Vehicle Bridge Replacements

As a result of raising Shasta Dam, the following local vehicle bridges would be replaced:

- Charlie Creek Bridge
- Doney Creek Bridge
- McCloud River Bridge
- Didallas Creek Bridge

Advance planning studies were prepared for the four bridges listed above (refer to Plates 27 through 30). The Second Creek Bridge, which was addressed in the PFR, is not included in the above list because the relocated replacement structure is anticipated to consist of a small culvert. Relocation costs for this bridge are included in the road relocations cost estimates.

Design Criteria and Assumptions

Criteria and assumptions considered in determining structure type and length for the replacement structures included the following:

1. Structure type based on cost and constructability.

2. Superstructure depth based on depth-to-span ratios of 0.04 for CIP prestressed concrete box girder structures, and .055 and 0.06 for multispan and single-span CIP reinforced-concrete box girder structures, respectively.

3. Four-foot minimum freeboard above full pool elevation 1,087.50 (18.5-foot dam raise, NGVD29).

4. Approach fill height into the river/creek channel limited to approximately 10 feet to minimize reduction of hydraulic opening.

5. Total bridge width of 31 feet, 6 inches, based on two 12-foot lanes with 2-foot shoulders (low volume road) and 1-foot, 9-inch barrier railing widths.

6. Use of driven steel piles and sheet piling for foundations/falsework and cofferdams, respectively.

Bridge Replacement Summary

Based on the above noted design criteria and assumptions, and considering horizontal alignments and profile grades developed for the relocated roadways, Table 4-6 summarizes proposed bridge characteristics for the four vehicle bridges requiring replacement.

Table 4-6. Vehicle Bridge Replacement Summary Table

Bridge Name	Bridge Type	Bridge Length (feet)	Structure Depth (feet-inches)	Bridge Deck Profile Grade Elevation (feet)	Vertical Clearance (freeboard) Above Full Pool (feet)
Charlie Creek Bridge	CIP Prestressed Concrete Box Girder	782	10-0	1,102.0	4.5
Doney Creek Bridge	CIP Prestressed Concrete Box Girder	760	10-0	1,102.0	4.5
McCloud River Bridge	CIP Reinforced-Concrete Box Girder	490	8-3	1,100.25	4.5
Didallas Creek Bridge	CIP Reinforced-Concrete Box Girder	115	7-0	1,099.0	4.5

Key:
CIP = cast in place

Construction Quantities

Construction quantities for major items of work are summarized in Table 4-7. Demolition of the four bridges would produce about 9,900 cubic yards of waste material.

Table 4-7. Construction Quantities for Vehicle Bridge Construction

Item Description	Charlie Creek Bridge	Doney Creek Bridge	McCloud River Bridge	Didallas Creek Bridge
Excavation (cubic yards)	1,200	550	820	440
Backfill (cubic yards)	480	400	530	180
CIP Structural Concrete (cubic yards)	3,530	3,320	2,320	760
Bar Reinforcing Steel (pounds)	1,124,000	1,006,000	757,000	208,000
Prestressing Steel (pounds)	26,000	25,000	N/A	N/A
Class 140 Piles (each)	24	24	24	24
Class 140 Piles (linear feet)	1,080	1,080	1,080	1,080
24-inch Cast-In-Steel-Shell Piles (each)	72	72	32	N/A
24-inch Cast-In-Steel-Shell Piles (linear feet)	3,600	3,600	1,600	N/A

Key:
CIP = cast in place
N/A = not applicable

Fender’s Ferry Bridge Modifications

The Fender’s Ferry Bridge is a three-span structure comprising a steel plate girder superstructure supported on riveted steel tower bents and reinforced-concrete piers with spread footings. As a result of differences in West River and East River bank topography, the Pier 3 steel tower is supported at a much lower elevation than the Pier 2 tower. Thus, considering a full pool elevation of 1,087.50 (NGVD29), the Pier 3 steel tower would be inundated.

Proposed Modifications

A preliminary modification concept previously proposed consisted of constructing a concrete box extension to protect the existing steel tower (Reclamation 2004a). However, considering the narrow geometry of the concrete pier and steel tower, construction of a concrete box around the existing steel tower would preclude adequate future inspections and likely result in maintenance problems. Thus, it was recommended that the existing reinforced-concrete pier and footing be enlarged and extended, and the existing steel tower be modified to prevent inundation as a result of the higher joint-use pool level Proposed modifications would include the following:

- Enlargement of the existing reinforced-concrete footing
- Enlargement and extension of the existing reinforced-concrete columns and pier wall to elevation 1,096.16 (NGVD29)
- Removal of approximately 24 feet of the lower portion of the Pier 3 steel tower (based on location of existing cross bracing)
- Reuse of the existing steel bearing assemblies

Construction activities would likely be completed from the existing embankment without the need to construct cofferdams around the pier because the average water surface elevations are below the existing Pier 3 bottom of footing elevation for all months, with the exception of April and May. Construction of temporary bents to support the superstructure would be necessary to facilitate construction of the pier modifications. During construction activities, temporary traffic controls may be needed to facilitate delivery of materials and construction of temporary support bents. Refer to Plate 31 for the Advance Planning Study illustrating the proposed modifications.

Construction Quantities

Construction quantities for major items of work are summarized in Table 4-8.

Table 4-8. Fender's Ferry Bridge Construction Quantities

Item Description	Quantity
Excavation (cubic yards)	75
Temporary Superstructure Support (lump sum)	1
Concrete Surface Preparation (square feet)	2,150
Drill and Bond Dowels (each)	540
Structural Concrete (cubic yards)	230
Bar Reinforcing Steel (pounds)	66,400
Removal of Portion of Existing Steel Tower (lump sum)	1
Structural Steel (pounds)	130
Lead Paint Containment (lump sum)	1

Union Pacific Railroad Bridge Replacements

As a result of the raising of Shasta Dam, the following UPRR bridges would need modification or replacement:

- Pit River Bridge
- Sacramento River 2nd Crossing Bridge
- Doney Creek Railroad Bridge

Pit River Bridge Pier Modification

The existing bridge was designed and built by Reclamation in 1938 as part of the relocated highway and railroad facilities required for the construction of Shasta Dam. The bridge is still owned by Reclamation. Plate 32 is a copy of the original design drawing showing the bridge plan, elevation, and sections. The bridge is a multipurpose structure, carrying both UPRR and Interstate 5 traffic. The bridge is both a steel-through truss and a deck truss. UPRR and Caltrans have joint operation and maintenance responsibility. The bridge main structure is approximately 2,754 feet long; including the approach spans, it is approximately 3,588 feet long. The new top of full pool elevation was set based on providing a minimum 4-foot freeboard below the existing Abutment 2 bearing seat elevation.

The elevation at the top of existing Pier 3 concrete is 1,069.67 (NAVD88). (Note that the following elevations presented in this section are based on the NGVD88 datum). This elevation matches the existing top of joint-use (full pool) elevation of 1,069.67. The elevation at the top of existing Pier 4 concrete is 1,072.19, which is 2.52 feet above the existing top of joint-use (full pool) elevation. The new full pool elevation would be 1,090.2, which would inundate the existing bridge bearings and low-chord steel truss members. The remainder of the Pit River Bridge structure would not be affected by the proposed dam raise. To keep the existing steel bearings and lower portions of the steel truss members from being submerged, a watertight concrete tub structure would be required. This reinforced-concrete structure would be attached to the top of the existing concrete Piers 3 and 4, as shown in Plate 33. The structure footprint is rectangular and is approximately 151 feet long by 52.5 feet wide. From the edge of the existing pier, the interior base of the tub extends 8 feet and the side slopes approximately match the slope of the existing steel truss members. Structure thickness varies from 2 feet to 8 feet. The top of the concrete structure is set to be 4 feet above elevation 1,090.2. Four-inch-diameter holes would be drilled through the existing concrete for installation of bundled No. 11 bars to anchor the structure to the existing pier.

Piers 3 and 4 Protection Sump Pumps. Since the existing bridge superstructure and top of pier are exposed to the elements, a structure cover would not be required; however, sump pumps would be installed that would keep any water away from the bearings. The following assumptions have been made:

- Existing highway drains would be redirected away from the sump
- 120-volt (V) alternating current would be available
- Telephone line would be available for the requested alarm system
- Railroad tracks would be built on open style supports that would allow passage of water
- Two 2.5-by-2.5-foot sumps in the concrete would be provided

Two submersible sump pumps would be used to keep the water level in the new concrete protective structure from rising near the bearings. Each pump would discharge into 2-inch-diameter copper tubing, and the two lines would tee into a 2.5-inch-diameter line that would follow the slope upward to the discharge point. Check valves and ball valves would prevent pumped water from draining out of the line back into the sump, and would isolate the sump. Protective grates would prevent large objects from entering the sump area. A high-water alarm would be used to alert personnel someone if the pumps did not function properly. An electrical engineer would provide power and alarm designs. Brief research showed a 2006 peak rainfall in Redding, California, of 5 inches per hour. The pumps were sized for this peak level even though the tub structure would be partially covered because winds and other drainage problems could cause larger amounts of water to enter the sump. It was assumed that the current bridge drains would be redirected to discharge outside the protective structure. If the bridge drains cannot be redirected, larger pumps may be needed. Construction quantities for major items of work for this feature are summarized in Table 4-9.

Table 4-9. Pit River Bridge Modifications Construction Quantities

Item	Quantity
Concrete (yd^3)	4,000
Reinforcing Steel (lbs)	1,200,000
Core Drilling (lf)	2,200

Key:
lbs = pounds
lf = linear feet
yd^3 = cubic yards

More details regarding Pit River Bridge modification designs are contained in Reclamation *Technical Memorandum No. SHA-8140-FEAS-2007-1* (2007i).

Union Pacific Railroad Bridges

The existing Sacramento River 2nd Crossing and Doney Creek railroad bridges were designed and built by Reclamation, and are operated and maintained by UPRR. The bridge superstructures consist of deck truss bridges with a single track, and the piers and abutments were designed to accommodate a future parallel single-track superstructure. Portions of both bridges would be submerged for any reservoir raise and would need to be replaced with new higher superstructures. Structural analyses of the existing bridge piers under design earthquake loads indicated that new bridge piers would be required. Minimal changes would be required for the railroad vertical alignment. The feasibility designs would permit uninterrupted rail service during construction.

For this feasibility study, the bridge superstructures and substructures were designed to accommodate a single track according to the American Railway Engineering and Maintenance of Way (AREMA) code (2007). The UPRR office in Omaha, Nebraska, indicated a preference for a deck girder superstructure rather than a through-truss or girder for both replacement bridges. The proposed new bridge superstructure would be a composite superstructure consisting of steel plate girders and a reinforced concrete deck. In general, the bridge superstructures would be designed to be continuous over the piers. However, with a requirement for 16 feet of vertical clearance underneath Span 2 for the Sacramento River 2nd Crossing Bridge, with a minimum width of 30 feet, to allow for the passage of houseboats, Span 2 is a simply supported span. No minimum clearance for houseboat traffic would be required for the Doney Creek railroad bridge. Large-diameter concrete columns with drilled shafts would support the superstructure and be founded on bedrock. The Sacramento River 2nd Crossing railroad bridge would require nine spans with a total length of 982 feet between concrete abutments (see Plates 34 and 35). The Doney Creek railroad bridge would require five spans with a total length of 537.5 feet between concrete abutments (see Plates 36 and 37). Construction quantities for major items of work for these features are summarized in Table 4-10.

Table 4-10. Railroad Bridge Construction Quantities

Item	Sacramento River 2nd Crossing Bridge Quantities	Doney Creek Bridge Quantities
Steel Truss Bridge Removal (lbs)	3,300,000	2,000,000
Concrete Removal (yd^3)	15,310	4,570
Excavation ($yd^{3)}$	2,100	630
Backfill (yd^3)	1,900	2,200
Concrete , including Shafts (yd^3)	11,700	7,080
Reinforcing Steel (lbs)	3,420,000	1,760,000
Drilled Shafts, 6-foot diameter (lf)	130	230
Drilled Shafts, 8-foot diameter (lf)	110	N/A
Drilled Shafts, 12-foot diameter (lf)	120	N/A
Drilled Shafts, 14-foot diameter (lf)	N/A	416
Drilled Shafts, 16-foot diameter (lf)	280	N/A
Structural Steel in Girders (lbs)	4,750,000	2,250,000

Key:
lbs = pounds
lf = linear feet
N/A = not applicable
yd^3 = cubic yards

More details regarding the Union Pacific Railroad Bridge designs are contained in Reclamation *Technical Memorandum No. SHA-8140-FEAS-2007-1* (2007i).

The proposed relocation of the railroad bridges would require realigning the railroad tracks between the two bridges. This realignment would parallel the existing tracks with a 25-foot offset to the east. Proposed horizontal and vertical alignments for the new railroad tracks between the two new railroad bridges are shown in Plate 38. Construction quantities for major items of work for the railroad realignment between the UPRR bridges are summarized in Table 4-11.

Table 4-11. Railroad Realignment Construction Quantities

Item	Railroad Realignment Between Bridges
Removal of Existing Railroad Track (tons)	370
Excavation (yd^3)	35,000
Compacted Backfill (yd^3)	7,500
Railroad Track (tons)	390
Concrete Railroad Ties (each)	4,200
Ballast (tons)	26,500

Key:
yd^3 = cubic yards

Recreation Facilities

The Whiskeytown-Shasta-Trinity NRA was established November 8, 1965, by Congress to provide for public outdoor recreation use and enjoyment. The NRA offers a variety of outdoor activities, including boating, water-skiing, swimming, fishing, camping, picnicking, hiking, and hunting.

Shasta Dam and Reservoir are located in the Shasta Unit of the NRA. Shasta Reservoir and is the largest man-made reservoir in California, with 370 miles of shoreline, and a surface area of 29,600 acres, making the lake ideal for recreation.

Any raise of Shasta Dam would have some effect on the many recreation features found along the reservoir shoreline. These features include marinas/boat ramps, resorts, campgrounds/day use areas, trails, and USFS facilities. Reclamation would protect such facilities from inundation, modify existing facilities to replace affected areas (i.e., relocate facilities on site) or abandon existing facilities and replace them at other suitable sites (i.e., relocate facilities off site).

Although USFS has not approved relocation sites or recreation site plans, preliminary mitigation plans for effects of an 18.5-foot dam raise on Shasta Lake recreation facilities have been developed with the cooperation of USFS. Plate 39 details the location of potential recreation mitigation areas (referred to as windows) associated with an 18.5-foot dam raise; existing recreation sites with proposed modification, expansion, or relocation; and proposed new recreation sites. (Preliminary mitigation plans have not been developed for lower dam raise heights, but would likely require fewer mitigation areas and relocations.) After authorization of the project, further detailed designs would need to be developed. The primary goal of the relocation plans is to verify that with any dam raise, the existing recreation capacity could be maintained. Reclamation and USFS would continue to work together to revise a recreation plan that is suitable for the NRA.

Decisions about whether individual affected facilities would be modified or relocated would be addressed in conjunction with USFS, based on overall effects on the features of individual facilities as well as operational needs. Some relocated facilities may be consolidated within other existing facilities, rather than being relocated at a currently undeveloped area. All plans for replacing of facilities would be evaluated and approved by USFS.

Where feasible, Reclamation would protect recreation facilities from seasonal high-water levels by installing retaining walls or similar structures to prevent inundation. The surface level of affected paved and unpaved areas (most used for parking) would be raised if that would prevent inundation. In areas where this would not be feasible or would be impractical, new facilities to replace lost parking areas would be constructed in adjacent unaffected areas.

All capacity of recreation facilities (e.g., boat launching, campsites, picnic sites, marina moorage and related services, resort lodging) lost as a result of inundation would be replaced. Reclamation would seek to maintain the quality of visitor experiences by replacing affected recreation facility capacity with facilities providing equivalent visual resource quality, amenities, and access to Shasta Lake and terrestrial natural resources.

Inundated recreation facilities and associated utilities would be relocated before demolition, with the exception of facilities identified for abandonment. Proposed comprehensive plans would, at minimum, maintain the existing recreation capacity at Shasta Lake. Recreation facilities proposed for relocation are included in the description of each comprehensive plan below. Construction-specific information regarding relocation and demolition of recreation facilities is under development and will be completed after the SLWRI is authorized.

Marinas/Boat Ramps Modifications

Several marinas around Shasta Lake would be affected by raising Shasta Dam. Typically, marinas consist of a parking area, a boat ramp, various structures (e.g., retail, restrooms, maintenance facilities, storage, administration), and utilities (power, water, and septic). Most of the effects of the dam raise would be due to the inundation of boat ramps, parking lots, structures, and utilities. Boat ramps would be modified in place on fill, where possible. Modifications to parking areas would include replacing them on fill, or relocating them above the new reservoir elevation. Existing structures that would be inundated would be demolished, and either replaced above the reservoir elevation (upslope or on placed fill), or moved to a floating structure on the water to provide better access for recreational users. Any access roads would be relocated above the new full pool to continue to provide access around the marinas. Existing septic systems that would be inundated would be demolished and removed from the area or relocated. New facilities could also be connected to new localized wastewater treatment facilities. Power lines would be installed to accommodate new structures. Marinas and boat ramps that could not be modified in place would be relocated to adjacent areas that are capable of providing the necessary grade and access for ramps, or facility capacity would be replaced at other facilities identified to have potential for expansion. To maintain current recreation capacity, as much as 10.7 acres of expanded, or new, boat ramp and/or marina land use would be needed. The following potential areas could be used to meet this need:

- Antlers Boat Ramp and adjacent marina area
- Packers Bay Marina
- Silverthorn Marina Area
- Holiday Harbor

See Plate 39 for locations of these potential recreation areas.

Resorts Modifications

Raising Shasta Dam would affect as many as four resorts around the reservoir to some degree. Inundated structures, and those within 3 feet of the new full pool, would be demolished. Associated septic systems would also be demolished, and the remaining structures would either connect to new localized wastewater treatment facilities or be relocated to other septic systems. As much as 14 acres of expanded, or new, resort land use would be needed to maintain current recreation capacity.

Campgrounds/Day Use Areas Modifications

Several undeveloped areas have been identified as potential campgrounds to replace capacity lost because of inundation. While all, or portions of some, inundated campgrounds would be relocated at their existing location, others would be moved around the reservoir to new locations identified as potential campground sites. As much as 30 acres of expanded, or new, campground area would be needed to maintain current recreation capacity. The following areas could be used to meet this need:

- Antlers Campground
- Oak Grove Campground
- Hirz Bay Campground
- McCloud Bridge Area

As much as 6 acres of expanded, or new, boat-in campgrounds would be needed to maintain current recreation capacity. The following areas could be used to meet this need:

- Lakeview Marina Area
- Monday Flat Boat-In Camp

As much as 6 acres of expanded, or new, day-use land use would be needed to maintain current recreation capacity. The following areas could be used to meet this need:

- Ellery Creek Campground
- Gregory Creek Campground
- McCloud Bridge Area
- Upper Salt Creek

See Plate 39 for locations of these potential mitigation areas.

Recreation Trails

Portions of most Shasta Lake trails would be affected by any dam raise. Affected segments of hiking and biking trails would be relocated upslope to restore the continuity of affected trails. As much as 11.6 miles of trails and 2 trailheads would need to be relocated. In addition, CP5 includes construction and/or modification of existing facilities at various locations to provide for 18 new miles of trails and 6 trailheads to enhance recreation opportunities at Shasta Lake.

USFS Facilities Modifications

Recreation within the NRA is managed by USFS, which has several facilities located throughout the reservoir area. USFS facilities consist of various storage and maintenance buildings and equipment, fire protection equipment, customer service facilities, office space, and employee living facilities. Two USFS facilities would be inundated at all dam raise heights, and require relocation or replacement: Lakeshore Fire Station and Turntable Bay Station. Lakeshore Fire Station would be relocated to an area above the new full pool in the same general vicinity of the lake, providing necessary access to Interstate 5 and minimizing potential conflicts with adjacent recreation, residential, and commercial areas. The new facility would contain all of the features that exist at the current facility. The inundated facility would be demolished, and hauled to waste. Additional space at Turntable Bay Station would allow this facility to be relocated on fill in its current location.

Access Roads

Reclamation's mitigation plans for recreation facilities include mitigation of project effects on roads and bridges (as described in the preceding discussion of roadway relocations), many of which are used to access recreation facilities. Facility access roads may be relocated, raised, or abandoned. If abandoned roads serve a substantial recreation-access purpose, mitigation may take the form of upgrading alternative access routes that serve the same areas.

Nonrecreation Structures Demolition

All nonrecreation structures subject to demolition must file a Demolition Declaration with the Shasta County Department of Resource Management's Building Division, pursuant to Section 19827.5 of the State Health and Safety code. Structure demolitions associated with this public project would require filing an Asbestos National Emission Standards for Hazardous Air Pollutants (NESHAP) Notification of Demolition and Renovation. An asbestos survey must be completed by a licensed and appropriately registered contractor, and the completed Asbestos NESHAP Notification must be sent to the U.S. Environmental Protection Agency (EPA). Any asbestos abatement would be as required by the agency. A copy of the Asbestos NESHAP Notification must be attached to the Demolition Declaration filed with Shasta County.

Grading or excavating activities associated with the structure demolitions for public works projects are typically exempt from the Shasta County Department of Resource Management's Environmental Health Division (EHD) Grading

Permit under Shasta County Code 12.12.050. An Application for Grading Permit, however, must be filed with EHD to indicate the exemption.

Structure Demolition Activity Description

Structure demolition would be performed by appropriately licensed contractors. All utilities would be disconnected, capped, and/or removed per permit requirements and governing utility standards. The structure and foundation would then be demolished. Asbestos material, if discovered, would be removed and taken to an approved landfill for disposal per permit requirements. General demolition waste would also be removed and trucked to an approved landfill.

A typical structure demolition crew would include an excavator with operator, a 50- to 70-yard end-dump truck with operator, a bobcat or similar small front-loader with operator, and two laborers. A typical house would reduce to about 3 to 4 feet high (5 to 6 feet for two-story houses) of fluffed material within the foundation footprint. Typical foundation construction is assumed to be a concrete stem wall around the perimeter of a structure with an overall height of 5 feet and thickness of 1 foot. Table 4-12 shows the total volume of demolished material by dam raise. Costs associated with nonrecreation structures demolition are shown in the utilities cost estimate in Chapter 5.

Table 4-12. Total Volume of Demolished Nonrecreation Structures

Dam Raise	Total Volume of Material (cubic yards)
6.5 feet	8,710
12.5 feet	21,450
18.5 feet	26,960

Ecosystem Restoration

CP4 and CP5 include ecosystem restoration measures around Shasta Lake and along its tributaries, as well as downstream from Shasta Dam along the upper Sacramento River.

Reservoir Area

Shoreline enhancement and tributary aquatic habitat enhancement are only considered reservoir area ecosystem restoration measures for CP5.

Shoreline Enhancement

The ecosystem enhancement goal for the shoreline environment of Shasta Lake is to improve the warm-water fish habitat associated with the transition between the reservoir’s aquatic and terrestrial habitats. Shoreline enhancement entails the range of enhancement opportunities along the Shasta Lake shoreline below the full-pool elevation of 1,090.2 (NAVD88) that would occur with an 18.5-foot dam raise. This area is typically between 0.1 and 1.5 miles upslope from the current full-pool elevation of 1,069.7 (NAVD88). The shoreline is defined as the area encompassing nearshore aquatic habitat within the reservoir itself and vegetation and other habitat components adjacent to the reservoir.

Two categories of potential nearshore warm-water fish habitat enhancement activities are discussed below: (1) *structural enhancements*, which entail construction and placement of artificial structures in Shasta Lake's littoral zone; and (2) *vegetative enhancements*, which entail planting and seeding to provide submerged and partly submerged vegetative cover when the reservoir is at full-pool capacity during the winter/spring months.

Structural enhancements associated with CP5 include placement of brush structures constructed from whiteleaf manzanita (*Arctostaphylos manzanita)* in Shasta Lake's littoral zone. Because of manzanita's density, installation does not require using anchor or cabling techniques that could result in ancillary negative impacts (e.g., maintenance, hazards to boaters). The brush structures would be assembled in the draw-down zone of the reservoir in an area that would be inundated as the reservoir surface elevation rises in fall. The brush structures are expected to be about 1,800 cubic feet in size. The establishment period would be the first year after construction; life span of the brush structures is projected to be 10 years.

Table 4-13 identifies the general area, number, and size of proposed structural enhancement locations for the main body of Shasta Lake, and the Pit, Sacramento, McCloud, Big Backbone, and Squaw arms. Selection of specific locations has been deferred so that enhancement locations would be consistent with other objectives of the SLWRI. The level of proposed treatment is based on the proportion of available manzanita surrounding Shasta Lake. In general terms, these locations would incorporate available material at locations with preferred topographic features; preferred locations are coves that offer steep drawdown areas during the primary use period (spring, early summer).

Table 4-13. Proposed Structural Enhancement of Lake and by Arm

Area	Area Treated (acres)	Number of Locations
Lake Main Body	17	595
Pit Arm	12	420
Sacramento Arm	43	1,505
McCloud Arm	8	280
Big Backbone Arm	3	105
Squaw Arm	17	595
Total	**100**	**3,500**

Vegetative enhancements associated with CP5 include planting willows (*Salix*) to enhance nearshore fish habitat, and aerial and hand seeding of annual cereal grains to treat shoreline areas at Shasta Lake.

Table 4-14 identifies the general area, number, and size of proposed vegetation enhancement locations for the main body of Shasta Lake, and the Pit, Sacramento, McCloud, Big Backbone, and Squaw arms. More than 30 acres could be available to enhance willow recruitment adjacent to Shasta Lake. Rooted willows would be planted in draws and other moist sites, such as springs, to provide long-term live cover. The establishment period for willows would be the first year after construction; life span is projected to be 5 to 50 years. The establishment period for cereal grains would also be the first year of construction, with the life span projected to be 1 to 3 years. This approach requires native seed and nursery stock; several years of advanced preparation would be needed before planting could take place.

Table 4-14. Proposed Vegetation Enhancement of Lake and by Arm

Area	Willow Planting (acres)	Native Grass Seeding (acres)
Lake Main Body	1	2
Pit Arm	1	4
Sacramento Arm	7	4
McCloud Arm	1	2
Big Backbone Arm	3	2
Squaw Arm	1	2
Total	**14**	**16**

Tributary Aquatic Habitat Enhancement

The quantity and quality of aquatic habitat in the tributaries of Shasta Lake are influenced primarily by the presence of road crossings and culverts, although in some cases, other structures or grade controls (e.g., transitional deltaic deposits) may constitute barriers to aquatic connectivity, including fish passage.

Barriers to fish passage in the watersheds above Shasta Lake are primarily associated with culverts or other types of stream crossings. Typical passage problems created by culverts and other road crossings include the following:

- Excessive drop at the downstream end of a crossing (perched outlet)
- Water velocities within a crossing that are too fast to allow fish to swim upstream
- Constriction of flow as it enters a crossing, causing excessive water velocities and turbulence at an inlet
- Lack of sufficient water depth in a culvert for the fish to swim
- Debris accumulation across an inlet or within a culvert

Surveys have identified opportunities to restore and/or enhance 14 perennial and intermittent stream crossings to improve fish passage. Based on information obtained in the surveys, these crossings meet one or more of the criteria for impaired fish passage. Table 4-15 identifies the sites by road section, the watershed in which they occur (arm of Shasta Lake), and the type and size of crossing, and characterizes problems identified at these sites.

Downstream from Shasta Dam

Gravel augmentation and side channel restoration are considered ecosystem restoration measures for CP4 and CP5 downstream from Shasta Dam.

Gravel Augmentation

Gravel suitable for spawning has been identified as a significant influencing factor in the recovery of anadromous fish populations in the Sacramento River. As part of CP4 and CP5, spawning-sized gravel would be placed at multiple locations along the Sacramento River between Keswick Dam and the RBDD.

Gravel augmentation would occur at one to three locations every year, for a period of 10 years, unless unusual conditions or agency requests precluded placement during a single year. This program, in combination with the ongoing Central Valley Project Improvement Act (CVPIA) gravel augmentation program, would help address the gravel debt in the upper Sacramento River, but the reach may continue to be gravel-starved into the future. Therefore, the gravel augmentation program proposed herein would be reevaluated after the 10-year period to assess the need for continued spawning gravel augmentation, and to identify opportunities for future actions or programs to do so.

Table 4-15. Culvert Replacement on Perennial and Intermittent Streams

Road/Site No.	Watershed	Type/Size of Crossing	Problems
FS 35N08 Sugarloaf Creek (Site 1)	Sugarloaf Creek	Culvert / 13.5-foot-diameter	Undersized, misaligned culvert, velocity/gradient barrier, eroding fill slope
FS 35N60 (Site 3)	McCloud	Culvert / 48-inch-diameter	Undersized culvert, velocity/gradient barrier, plunge pool, fill slope erosion
FS 35N60 (Site 4)	McCloud (Ellery Creek)	Culverts / 3 culverts 5-foot-diameter	Multiple culverts, velocity/gradient barrier, plunge pool, fill slope erosion
FS 35N60 (Site 5)	McCloud (Moore Creek)	Culvert / 72-inch-diameter	Culvert damaged, undersized, fill slope erosion
FS 35N17 (Site 6)	Salt Creek	Culvert / 36-inch-diameter	Undersized, shotgun outlet, velocity/gradient barrier, eroding fill slope
FS 35N17 (Site 7)	Salt Creek	Culvert / 24-inch-diameter	Undersized, shotgun outlet, velocity/gradient barrier, eroding fill slope
FS 35N08 (Site 11)	Sugarloaf Creek	Culvert / 48-inch-diameter	Undersized culvert, velocity/gradient barrier, fill slope erosion
FS 35N60 (Site 12)	McCloud Arm	Culvert / 24-inch-diameter	Damaged culvert, velocity/gradient barrier, fill slope erosion
FS 35N60 (Site 13)	McCloud Arm	Culvert / 24-inch-diameter	Damaged culvert, velocity/gradient barrier, fill slope erosion
FS 35N60 (Site 14)	McCloud Arm	Culvert / 18-inch-diameter	Undersized culvert, velocity/gradient barrier, fill slope erosion
FS 35N60 (Site 15)	McCloud Arm	Culvert / 18-inch-diameter	Undersized culvert, velocity/gradient barrier, fill slope erosion
FS 35N60 (Site 16)	McCloud Arm	Culvert / 18-inch-diameter	Damaged culvert, velocity/gradient barrier, fill slope erosion
FS 35N60 (Site 17)	McCloud Arm	Culvert / 36-inch-diameter	Undersized culvert, velocity/gradient barrier, fill slope erosion
FS 35N60 (Site 18)	McCloud Arm	Culvert / 24-inch-diameter	Damaged culvert, velocity/gradient barrier, fill slope erosion

On average, 5,000 to 10,000 tons of gravel would be placed each year, although the specific quantity of gravel placed in a given year may vary from that range. Gravel would be obtained as uncrushed, rounded river rock, free of debris and organic material, from local, commercial sources. To maximize the benefit to anadromous fish, gravel would be washed and sorted to meet specific size criteria. To minimize impacts to salmonid spawning activity, gravel applied to active river channels would be placed between August and September each year, consistent with the time frame for the ongoing CVPIA gravel augmentation program.

Fifteen preliminary locations for spawning gravel augmentation were identified in the Sacramento River between Keswick Dam and Shea Island. Each site would be eligible for gravel placement one or more times during the 10-year program. Selection of these locations was based on potential benefits to anadromous fish and site accessibility. Gravel placement would provide either immediate spawning habitat or long-term recruitment.

Although preliminary sites have been identified, specific gravel augmentation site(s) and volume(s) would be selected each year in spring or early summer through discussions among Reclamation, the U.S. Fish and Wildlife Service, DFG, and the National Marine Fisheries Service. The discussions would include topics such as avoiding redundancy with planned CVPIA gravel augmentation activities in a given year; identifying hydrology or morphology issues that could impact the potential benefit of placing gravel at any particular site; identifying changes in spawning trends due to previous years' gravel augmentation activities; evaluating potential new sites; and are appropriately distributing selected gravel sites along the river reach(es).

Restore Riparian, Floodplain, and Side Channel Habitat

Under CP4 and CP5, riparian and floodplain habitat restoration would be constructed at a suitable location along the Sacramento River. The exact size, scope, and location of a suitable restoration site is still under development and a description of potential riparian, floodplain, and side channel habitat restoration at Reading Island is provided below as an example restoration project. Restoration activities anticipated under CP4 and CP5 are expected to be similar in size and scope to those described below.

Reading Island lies along the Sacramento River just north of Cottonwood Creek in Shasta County at River Mile 274. Reading Island is approximately 269 acres in area, with 46 acres on the south end of the island owned by the U.S. Department of the Interior, Bureau of Land Management (BLM) and managed as a day-use park (Figure 4-2). The remaining 223 acres are privately owned. The island is accessible by Adobe Road and a bridge crossing over the Anderson Creek Slough into the BLM day-use park. Historically, the channel that now forms the slough supported important habitat for anadromous salmonids, including rearing habitat for winter-run Chinook and spawning habitat for Central Valley steelhead.

At the Reading Island site, an approximately 0.8-mile-long historic Sacramento River channel/floodplain scour channel/side channel (hereafter referred to as "side channel") drains into the present-day Anderson Creek, a remnant Sacramento River side channel. Anderson Creek flows approximately 1.5 miles and then enters the Sacramento River about 0.3 miles upstream from Cottonwood Creek. Average channel width of the side channel is approximately 30 feet.

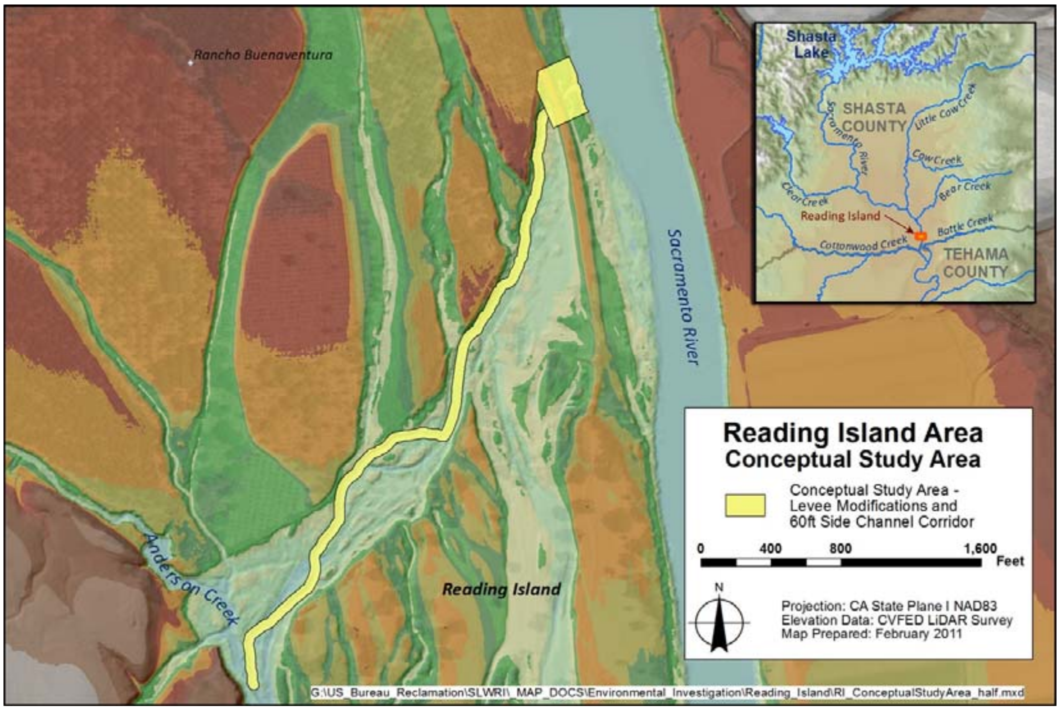

Figure 4-2. Reading Island Conceptual Study Area

The Anderson Creek Slough, into which Anderson Creek empties, was blocked at the upstream end in the early 1970s by construction of a levee on the adjoining private property. A few years after construction of the levee, the slough became choked with various species of water plants, primarily primrose creeper (*Ludwigia peploides).* Before levee construction, the Anderson Creek Slough captured a portion of the Sacramento River flow and functioned as side channel habitat.

After levee construction, water velocity in the side channel slowed substantially and water temperatures increased. Primrose creeper and warm-water nonnative fish species established within the channel. Currently, most of the water entering the slough comes from Anderson Creek and drainage wastewater from irrigation canals. An earthen embankment with two 36-inch-diameter culverts now restricts the flow of water into the side channel. The water surface elevation of the Sacramento River, with a flow rate of 8,500 cfs, is at the approximate elevation of the invert of the culverts, but even when discharge in the Sacramento River increases to approximately 12,000 cfs, there is minimal flow through the culverts into the side channel. Above the slough, Anderson Creek is known to provide rearing habitat for winter-run Chinook, and is managed for steelhead spawning habitat.

Floodplain, riparian, and side channel habitat restoration would involve acquiring property on Reading Island and revegetating floodplain terraces and adjacent riparian areas with native plants. In addition, the Reading Island side channel could be activated over a wider range of flows to provide juvenile salmonid rearing habitat in the side channel, and in Anderson Creek at the downstream end of the side channel. This would be accomplished by breaching the levee at the upstream end of the side channel to restore connectivity with the Sacramento River at flows greater than 4,000 to 6,000 cfs. Preliminary analysis indicates that in addition to breaching the levee, side channel clearing and excavation may be necessary to restore flows capable of supporting suitable spawning habitat. This would include vegetation and debris removal and deepening the existing channel. At a maximum, side channel clearing and excavation would be performed along the entire 0.8-mile channel, requiring the removal of about 15,560 cubic yards of material.

Planting mix, composition, and density would be determined by a more detailed site analysis, but could include native cottonwood, willow, box elder, valley oak, western sycamore, elderberry, and a variety of understory brush species. Temporary irrigation would be provided on an as-needed basis with a temporary well powered from an existing nearby power supply. The revegetated areas are expected to develop into self-sustaining riparian habitats within 1 to 4 years of initial planting, based on results of previous riparian restoration projects along the Sacramento River. Regraded floodplain areas are expected to change over time depending on hydrologic conditions, but it is anticipated that no elements of this measure would need to be replaced or reapplied during the 50-year project life. The site would be fenced to reduce the potential for access by livestock.

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Chapter 5
Opinion of Probable Construction Cost

Feasibility-level most-probable construction cost estimates for the Shasta Dam and Reservoir raise include 3 dam raise alternatives, resulting in 5 comprehensive plans with 19 to 23 separate features. The three dam raise alternatives considered raises of 6.5 feet, 12.5 feet, and 18.5 feet, respectively. Significant features were included separately related to the dam and reservoir raise. The cost estimates were intended to capture the most current pricing for materials, wages and salaries; accepted productivity standards; and typical construction practices, procurement methods, current construction economic conditions, and site conditions for the current level of design. The cost estimates were prepared with less than complete designs and have inherent levels of risk and uncertainties. The cost estimates are intended to be a basis for budget authorization, appropriation, and funding.

Feasibility-Level Cost Estimates

Feasibility-level cost estimates are based on information and data obtained during design investigations. These investigations provide sufficient information to permit the preparation of preliminary layouts and designs from which approximate quantities for each kind, type, or class of material, equipment, or labor may be obtained. These estimates are used to assist in the selection of a preliminary proposed plan to determine the economic feasibility of a project and to support seeking construction authorization from Congress.

The feasibility estimate unit prices were developed using a semi-detailed method. Specific construction activities were identified for major cost drivers. Costs for labor, equipment, materials, and other resources were developed. Production rates, overhead, and taxes were applied to develop applicable unit prices. Vendor quotations were obtained for major equipment, supplies, and other items. Minor cost items were developed using historical bid and industry standard reference cost data.

Major Cost Estimate Components and Assumptions

The assumptions listed for direct cost line items, and specifically factors used to determine indirect costs, are critically important to the overall accuracy of the estimate, and should be reviewed and understood by all parties.

Competitive Market Conditions at Time of Bid Tender

Estimates assume that Builder's Risk Insurance would be available to the contractor. If Builder's Risk Insurance is not available to the contractor because of the scope, security implications, or magnitude of the project, increased bid margins can be expected because the contractor would need to assume additional risks.

Price Level

All prices shown in the feasibility-level cost estimates are in April 2010 dollars.

Cost Estimate Allowances

Depending on the level of study, it is often impractical to identify all items associated with a project. Accordingly, appraisal, feasibility, and partial design estimates should contain a percentage allowance shown as a separate line item for unlisted items. This unlisted items allowance represents the amount required to achieve comparability between preliminary estimates and prevalidation estimates. In general, the less refined the estimate, the higher the percentages used. As more details are developed to refine a specific cost estimate, the number of direct cost line items increases, the accuracy of the quantity takeoffs increases, and the allowance for unlisted items decreases.

Mobilization

A value of 5-10 +/- percent was used for mobilization. Mobilization costs include contractor bonds, and mobilizing contractor personnel and equipment to the project site during initial project setup. The assumed 5-10 (+/-) percent value in the cost estimates is based on past experience of similar jobs.

Design Contingency

A value of 10-20 +/- percent was used for design contingencies. Design contingencies are intended to account for three types of uncertainties inherent as a project advances from the planning stage through final design, which directly affects the estimated cost of the project. These include (1) minor unlisted items, (2) minor design and scope changes, and (3) minor cost estimating refinements. Based on the completeness of the listed items that the detail provided, the design contingency was set at 10-20 +/- percent of the listed items for this project, depending on the feature.

Allowance for Procurement Strategies

The allowance for procurement strategies (APS) was set at 2 percent. A line item APS (considerations) may be included in feasibility-level cost estimates to account for additional costs when solicitations will be advertised and awarded under other than full and open competition. These include solicitations that will be set aside under socioeconomic programs, along with solicitations that may limit competition or allow award to other than the lowest bid or proposal.

The Shasta Dam and Reservoir raise estimates assume full and open competition, receipt of sealed bids, with award to the lowest responsive and responsible bidder.

Construction Contingency

A value of 10-20 +/- percent was used for design contingencies for the majority of the features. The bridge features used a slightly higher value of 25 +/- percent. Feasibility estimates include a percentage allowance for construction contingencies as a separate item to cover minor differences in actual and estimated quantities, unforeseeable difficulties at the site, changed site conditions, possible minor changes in plans, and other uncertainties during the construction period. The allowance is based on engineering judgment of the major pay items in the estimate, reliability of the data, adequacy of the projected quantities, and general knowledge of site conditions.

Non-Contract Costs

Non-contract costs were estimated to be 32 ± percent of the total field costs based on typical non-contract cost percentage ranges from past large Reclamation projects. Land acquisition or relocation of property by others is not included in this percentage.

Non-contract costs include some of the following (this list is not all-inclusive):

- Environmental mitigation (10 percent of total field cost excluding environmental restoration).
- Cultural resources preservation (2 percent of total field cost).
- Planning, engineering design, and construction management. This includes collection, assembly, analysis of data, and preparation for environmental impact reports and surveying. This also includes construction designs and specifications, construction engineering and management, other costs such as general office salaries, supplies and expenses, general transportation expenses, security, environmental oversight, and legal services (20 percent of the total field cost).
- Land acquisition (see Real Estate Appendix for detailed analysis).
- Water use efficiency actions. This includes funding for an additional water conservation program for new water supplies created by the project, to augment current water use efficiency practices (see Environmental Impact Statement, Chapter 2, for more detail on the program).

Major Cost Estimate Exclusions

The feasibility-level cost estimates do not include costs associated with the following:

- Loss of water and power due to construction requirements affecting dam and powerplant operation
- Impacts to downstream water intakes
- General access road maintenance
- Impacts due to multiple construction contracts, market conditions, and number of bidders

Contractor Risks

Several risk items have been identified below in an effort to alert decision-makers to important issues that could impact contractor operations and costs:

- Wing dam and spillway modifications relative to fluctuating lake levels
- Schedule slippage due to security concerns
- Schedule delays for bird nesting restrictions
- Blasting operations at or near dam facilities
- Stringent classification of materials to meet specification requirements
- Material transport restrictions and safety concerns
- Processing areas that are identified as not sufficient to meet required production goals
- Insurance issues in relation to dam significance
- Seasonal work restrictions imposed by phased spillway gate and lower tier outlet gate replacement schedule
- Long contract periods that expose liabilities
- Contractual risk transfer
- Minority business enterprise and miscellaneous flow-down provisions

Escalation

An allowance for escalation from the April 2010 price level to the Notice to Proceed milestone was not included in the estimate. For projects that are to be developed over an extended period of time, or at some distant time in the future, it is prudent to consider the time value of money. Two distinct periods of time must be considered with escalation: (1) the period from the published price level until Notice to Proceed, and (2) the duration of the construction contract. The cost estimates only include escalation during construction, which is incorporated into the unit prices.

Since escalation through Notice to Proceed was not included, the legislation authorizing the construction of this project should contain appropriate language to provide Reclamation the authority to adjust the appropriation ceiling by indexing to reflect future changes in costs.

As mentioned, this estimate includes only first costs (without escalation from published price level to Notice to Proceed); however, escalation would be a very significant cost driver for the project. For economic analysis and future project budgeting, a preliminary escalation rate of 3 percent per year is recommended to be used.

The preliminary escalation rate is based on reliance of recent cost escalation data supplied by the Caltrans Price Index (Caltrans 2010). The Caltrans Price Index demonstrates that California recently experienced a significant downturn in prices for highway and heavy civil construction infrastructure work. Figure 5-1 shows Caltrans highway construction price trends from 1972 through April 2010, San Francisco City Cost Index trends from 1978 to April 2010 (Engineering News-Record 2011), and Caltrans highway construction price trends from 1972 through the present. From Caltrans Price Index data, expected long-term price trends have been projected through 2020.

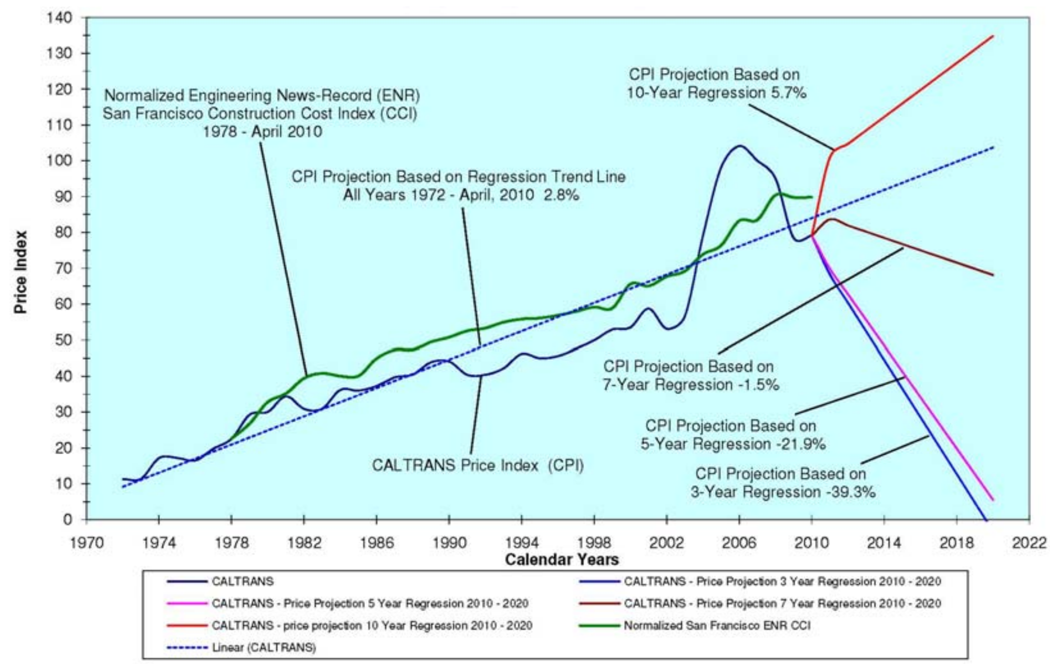

Figure 5-1. California Construction Price Trends

The construction market has experienced extreme price volatility in the last several years. A significant market anomaly occurring from 2002 through 2009 skews the calculation of forward cost trends using short-term linear regression techniques. Of note, the Caltrans Price Index decreased over 17 percent for the calendar year of 2008, and the Caltrans Price Index projection based on the 3-year regression indicates a negative average growth rate of almost 40 percent per year. Although the recent economic downturn has resulted in price decreases, it is expected that prices will continue to escalate over the long-term. While future inflation trends are difficult to predict, new market forces (e.g., higher material commodity pricing, energy costs, lack of competition) will likely continue to have significant impacts on heavy civil infrastructure construction costs for the foreseeable future. Because of uncertainty and variability among the short-term regressions, a longer view of the market is preferred. Consequently, while forward cost trends are always difficult to predict, there is some basis to believe that cost escalation is normalizing back to historical levels at approximately 3 percent per year. Future studies and coordination should be undertaken to determine the appropriate escalation factor to be used for budgetary approval.

Program Cost Drivers

Although not included in the estimates of first costs, escalation is a significant cost factor for the program and should be included for economic studies and future project budgeting. Total contingency is another significant cost driver. As explained previously, contingency consists of three separate components: estimating contingency, risk provision supported by probability theory, and unlisted items allowance.

Risk and Uncertainty

With each aspect of this report, certain assumptions were made based on engineering and scientific judgment. Careful consideration was given to the methodologies and evaluations for hydrology and system operations, cost estimates, and biological analyses. Analyses were developed with advanced modeling and estimating tools using historical data and trends. While this is an effective way to help predict outcomes for future operations, biological conditions, and costs, many uncertainties could affect the findings of this Engineering Summary Appendix. Various uncertainties and risks associated with the SLWRI are discussed in Chapter 5 of the Draft Feasibility Report.

Cost Estimates for Feasibility Report Alternatives

Estimated total construction costs and annual costs for each of the comprehensive plans are summarized in Table 5-1. More detailed cost estimate worksheets are included in the following attachments to this appendix:

- **Attachment 1 –** Cost Estimates for Comprehensive Plans
- **Attachment 2 –** 6.5-Foot Raise and Reservoir Area Infrastructure Cost Estimates

- **Attachment 3 –** 12.5-Foot Raise and Reservoir Area Infrastructure Cost Estimates
- **Attachment 4 –** 18.5-Foot Raise and Reservoir Area Infrastructure Cost Estimates
- **Attachment 5 –** Preliminary Construction Schedule and Work Packages
- **Attachment 6 –** CP4 Crystal Ball Estimate

The estimates of construction costs shown, and any resulting conclusions on the project's financial requirements, economic feasibility, or funding requirements, have been prepared from the best information available at the time the estimate was performed. Final project costs and resulting feasibility would depend on actual labor and material costs, competitive market conditions, and other variable factors, and should include escalation from the published price level to Notice to Proceed. Accordingly, the final project cost would vary from the estimate. Therefore, project feasibility, benefit/cost analysis, risk, and funding would need to be carefully reviewed before making specific funding decisions and/or establishing the project budget.

Table 5-1. Estimated Total Construction Costs for Comprehensive Plans

Item	CP1 6.5 Feet ($millions)	CP2 12.5 Feet ($millions)	CP3 18.5 Feet ($millions)	CP4 18.5 Feet ($millions)	CP5 18.5 Feet ($millions)
Field Costs					
Relocations					
Vehicular Bridges	$32	$32	$48	$48	$48
Doney Creek Railroad Bridge	$51	$51	$51	$51	$51
Sacramento River Railroad Bridge, Second Crossing	$105	$105	$105	$105	$105
Pit River Bridge Modifications	$15	$21	$28	$28	$28
Railroad Realignment	$7	$7	$7	$7	$7
Roads	$15	$23	$34	$34	$34
Utilities	$23	$24	$29	$29	$29
Buildings/Facilities - Recreation	$120	$135	$153	$153	$153
Dams and Reservoirs					
Main Dam	$49	$58	$69	$69	$69
Outlet Works	$25	$25	$25	$25	$25
Spillway	$95	$98	$100	$100	$100
Temperature Control Device	$26	$27	$28	$28	$28
Powerhouse and Penstocks	$1	$1	$1	$1	$1
Right Wing Dam	$4	$5	$6	$6	$6
Left Wing Dam	$12	$17	$23	$23	$23
Visitor Center	$8	$8	$8	$8	$8
Dikes	$13	$15	$23	$23	$23
Reservoir Clearing	$4	$7	$18	$18	$18
Pit 7 Dam and Powerhouse Modifications	$0.2	$0.2	$0.2	$0.2	$0.2
Environmental Restoration	-	-	-	$6	$17
Recreation Enhancement	-	-	-	-	$1
Total Field Costs	$605	$658	$757	$763	$764
Planning, Engineering, Design and Construction Management	$121	$132	$151	$153	$153
Lands	$26	$41	$60	$61	$61
Environmental Mitigation	$61	$66	$76	$76	$76
Cultural Resource Mitigation	$12	$13	$15	$15	$15
Water Use Efficiency Actions	$2	$3	$4	$2	$4
Total Construction Cost[1]	**$827**	**$913**	**$1,064**	**$1,070**	**$1,073**

Note:
[1] April 2010 price levels

Key:
- = not applicable
CP = Comprehensive Plan

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